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The Colorado River Siphon at Yuma, Arizona

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This article is a general description of the difficulties encountered in constructing an inverted siphon under the Colorado of the West, at Yuma, Ariz., preceded by a brief statement outlining the project of which the Siphon is a feature and giving some data on the physical characteristics of the river.

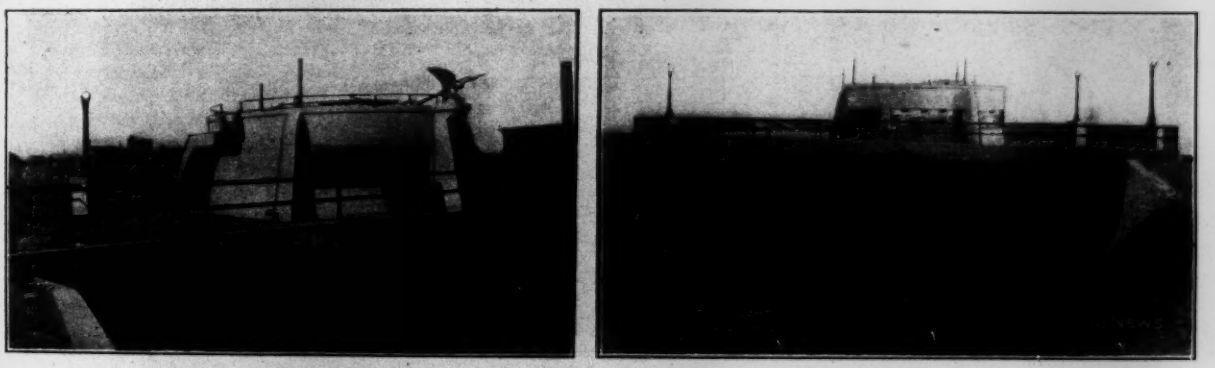
The Yuma Reclamation Project

The area now being prepared for irrigation by the U. S. Reclamation Service and known as the Yuma Project, lies along the Colorado River (which here separates California and Arizona), extending from the Mexican boundary to a point about 40 miles north thereof (Fig. 1). The land to be reclaimed from its present desert state comprises an ultimate area of approximately 130,000 acres unevenly divided between the above states. That in California consists of about 16,000 acres of bottom lands in the Yuma Indian Reservation in the extreme southeastern part of the state. The Arizona lands include some 55,000 acres of alluvial bottoms between the town of Yuma and the International boundary, 20,000 acres of similar lands in the Gila River Valley above Yuma, and 40,000 acres of mesa or table land south and east of the town. This latter area is about 80 ft. above the bottom lands and can be reached by pumps.

Following the passage of the Reclamation Act in June, 1902, investigations were made along the lower Colorado to determine the most favorable sites for irrigation works. These examinations, which covered several hundred miles of the lower portion of the stream, indicated that the area now embraced in the Yuma Project should receive attention first, and in 1903 the Secretary of the Interior made an allotment of funds for the inauguration of the work.

The scheme adopted, and now about 72% complete, provides for a diversion from the river, 14 miles above Yuma, at Laguna Dam (Eng. News, Feb. 27, 1908; June 10, 1909). Main canals leading from either end of the dam will supply the lateral systems serving the areas above outlined. The original design placed the larger diversion at the Arizona end of the structure, which arrangement involved a crossing of the Gila with an inverted siphon consisting of three 10-ft. tubes about 3000 ft. in length. The Gila, which enters the Colorado just above Yuma, is a stream that may be absolutely dry for months and then rise in a few hours to a discharge of 100,000 sec.-ft. or over. As it flows through alluvial soil, its banks have little stability and the river meanders at will during the sudden changes in its flow. Rapidly eroding banks are always present when there is appreciable discharge, and at times radical changes are caused by avulsion. The proposed siphon at the Gila was but a few miles below the diversion point, where any interruption in the flow of the canal would affect practically the entire Arizona area of the Project. It was essential that the crossing of this river should be safe beyond a reasonable doubt. The location selected was near the center of the valley and it appeared from a study of the meanders of the stream that a structure at this point would be seriously menaced by every flood of any magnitude. Investigations showed that there was a favorable site for a crossing several miles further upstream, where stable banks about 3000 ft. apart existed. A change in location to this point would involve heavy cuts in approaches and also materially lengthen the line.

Examinations of a main-canal location on the California side indicated that a favorable route existed without excessive depth of excavation. Such a route was about seven miles shorter than one on the Arizona side, and the conditions were such that over 1000 hp. of electrical energy could be generated from a drop in the canal. This route involved a siphon under the Colorado River which would be but one-third the length of a similar structure across the Gila and had the added advantage of crossing between fixed banks. While a siphon under the Colorado would be much deeper than one under the Gila, and expensive shafts would be necessary, in the final analysis it appeared that in addition to the opportunity for power development, the California route would cost less and was also free from the caprice of the river. Hence it was chosen. This canal from Laguna Dam to the Siphon has been constructed with a bottom width of 80 ft., a water depth of 7 ft., and a capacity of about 1400 sec.-ft.



Housing and Cylinder Gate; Yuma Siphon

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The Colorado River

In Bulletin No. 44 of the Arizona Experiment Station, Prof. R. H. Forbes thus describes the river:

The Colorado river is in many respects remarkably similar to the historic Nile. Like the Nile, it rises in a distant mountainous country; its lower courses traverse a subtropical and nearly rainless desert; and both rivers empty into great land-locked arms of the ocean at a little less than 32° north latitude. Like the Nile, the Colorado has for long distances along its lower courses created a narrow ribbon of fertile soil in the midst of the desert through which it flows, and has deposited a great alluvial delta between Yuma and the Gulf of California. This delta is chiefly Mexican territory. Like the great river of Egypt, the Colorado is subject to an annual summer rise sufficient to overflow great areas of its border and delta lands. These high waters are rich in fertilizing sediments, are excentionally free from alkaline salts, and come at an opportune time for irrigation. When the Colorado floods are less than normal, the Yuma Indians fail of satisfactory crops just as do the Egyptians with a low Nile. Although the climatic conditions in winter are slightly more severe in the Colorado Delta than in that of the Nile, yet these two regions closely resemble each other agriculturally. Some common products are (or may be) alfalfa, wheat ,and grains of the sorghum class; the date palm, fig, orange, olive ,and pomegranate; cotton, melons, and sugar cane.

When the Colorado is understood and utilized as successfully as its better known parallel, it will be recognized as the American Nile, the creator of a new country for the irrigator, and Mother of Occidental Egypt.

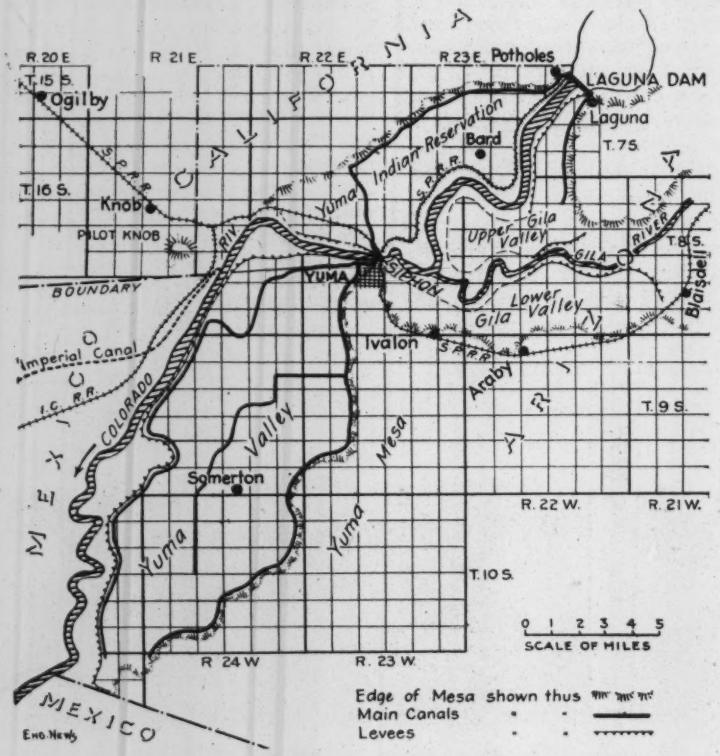


Fig. 1. Map of Yuma Reclamation Project at Yuma, Ariz., U. S. Reclamation Service

This river, with its never-failing supply, is formed in the southeastern part of Utah by the union of the Grand and the Green. The latter is the larger of the two and in reality the upward continuation of the Colorado. Including the Green, the Colorado is 2000 miles in length; its drainage ground is 800 miles long, varying in width from 300 to 500 miles, and contains approximately 300,000 square miles, ranking in extent as the second drainage area in arid America. This area embraces southeastern Wyoming, western Colorado, eastern Utah, practically all of Arizona, and some portions of California, Nevada, New Mexico, and Old Mexico. The snows on the mountains of Wyoming, Utah and Colorado furnish most of the supply for the rivers of the Colorado system. Flowing through deep cañons in their upper reaches, the silt-laden waters are preserved for the irrigation of the fertile bottom lands below.

During low stages the discharge at Yuma is from 3000 to 4000 sec.-ft., and in flood it may rise (as in 1909) to 150,000 sec.-ft. The freshet begins in March or April, when the melting snows are first in evidence, and the crest generally occurs in June, after which there is a rather sharp decline in gage height, comparatively low stages being reached in August and September.

The flood as observed at Yuma has one striking peculiarity. While the rise indicated on the gage from low water to crest is in the vicinity of 10 or 12 ft., there is an accompanying depression of the bed due to scour which, in seasons of maximum flood, may reach 30 or 35 ft. Thus the gage readings alone give no measure of the volume of discharge. The magnitude of this depression of the bed is well illustrated on the accompanying diagram (Fig. 2), which shows the flowing prism at low water in December, 1908, and the corresponding prism at the crest of the subsequent flood in June, 1909. The area of the prism increased from 1600 sq.ft. to 20,330 sq.ft., or 12.7 times. The maximum depth at low water was 5.4 ft. and at the crest 53.5 ft., but only 11.65 ft. of this increase was due to rise on the gage, the balance being the result of scour. The sections shown by dotted lines indicate how quickly the bed returned to low-stage conditions following the crest of the flood.

The Colorado is a silt-bearing stream of considerable magnitude which in the course of centuries has built a delta extending from its apex near Yuma to the Gulf of California. The depths of these alluvial deposits have never been approximately established except by borings in the river bed near Yuma, where it appears that soft sandstone occurs at a depth of about 80 ft. Above Yuma the river valley is from 1 to 6 miles in width, while the width of the channel is from 1000 to 4000 ft.; the slope of the water surface approximates 1.2 ft. per mile. Mean velocities measured by current meter at the Yuma gaging station range from 2 ft. per sec. at extreme low stages to 12 ft. per sec. at the highest floods. At half-flood stage the stream overflows its banks and inundates considerable areas on both sides; in fact, all of the bottom lands included in the Project have been subject to overflow at various times and levees are provided for their defense. Some 50 miles of levees are already built. On its lower reaches the stream is continually cutting its banks and changing its course, which action greatly augments the volume of silt carried and has meant failure or destruction for the private canals with small capital behind them which attempted to divert its waters. The above conditions have made much of the construction extremely difficult and place some of the work in a class by itself.

Crossing the River—Selection of Method

From what has been written and from Fig. 1, it is clear that while the main diversion for the project is made in California, the greater portion of the area to be irrigated is situated on the opposite side of the Colorado River in Arizona; therefore the main canal must be carried across the stream. The canal arrives at the west bank of the Colorado with its water surface at El. 132 above mean sea-level; having a water depth of 7 ft., the bottom of the canal is El. 125. The crests of the Colorado floods

at this point range from El. 125 to 134.

A crossing by continuing the canal at grade in a flume has been suggested. The canal has a capacity of 1400 sec.-ft. and a mean velocity of 2½ ft. per sec. Should the canal velocity be maintained in the flume, a structure with inside width of 80 ft. and depth of 7 ft. would be required. When full the flume would contain 560 cu.ft. per lin.ft., which, at 62.5 lb., is equal to 17 tons per lin-ft. for live load alone, making no allowance for the structure itself. Suppose by an increase in slope a velocity of 7 ft. per second was maintained in the flume, there would then be 200 cu.ft. of water per lineal foot when the structure was full, making the live load about 7 tons per foot of length.

The main line of the Southern Pacific now crosses the river at Yuma on a modern single-track steel bridge supported on concrete piers and abutments. This structure contains one span of 200 ft., two of 160 ft., and a short draw span at the Arizona end. The weight of this bridge is probably not far from 1½ tons per lin.ft. and is assumed to be safe for a live-load of 2 tons per lin.ft. The piers and abutments which support this structure, with its loading of 3½ tons per lin.ft., obstruct the waterway all that should be allowed. A former timber trestle at this point, supported on pile bents, was washed out. It appears, then, that the foundations for a structure carrying a live load which alone is double the combined live and dead load of the existing bridge, could not be placed in the river with safety.

But there is another condition besides obstruction of foundations which absolutely prevents the construction of a flume at this point. As shown above, the waterway of the flume would lie between El. 125 and 132, while the flood crests lie between 125 and 134, so that such a structure would completely block the upper portion of the channel at flood time. Furthermore, the bottom elevation of 125 is for the inside of the floor of the flume. When allowance is made for the thickness of the floor and its supporting girders, the extreme bottom will be at least as low as 122. Ordinary low water is 117, so a flume as outlined above would interfere with the flow of the river when a rise of 5 ft. had occurred, or at about half-flood stage. Under such conditions the maintenance of a canal structure crossing the river at grade is impossible. The crossing of the river should be accomplished in a manner that would leave the flowing prism unobstructed at all stages, and so an inverted siphon was decided upon.

The Siphon—General Description

The point selected for crossing the river is at the town of Yuma about 350 ft. downstream from the Southern Pacific Ry. bridge. Nine borings were made along the route selected, as shown on the profile, Fig. 3, and in each of them soft sandstone or indurated sand was encountered. Judged by the particles brought up in the wash borings, the sandstone is of the same general character as the bluff which flanks the eastern border of the river at this point. The borings showed this hard formation to dip toward the west, being about 50 ft. below the low-water bed in midstream and some 80 ft. under the surface on the site of the California shaft. Overlying the sandstone there is at low-water stages at least 25 ft. of very fine silt, which acts as a partially impervious blanket to the material below.

It was assumed, on account of this heavy blanket of overlying silt, that a tunnel well inclosed in the sandstone would encounter comparatively little water and might be driven without the aid of compressed air. To accomplish such an object, it would be necessary to sink shafts well into the lower and harder stratum, seal them against the water from above, and then cut through the shaft on line of the tunnel. It is reported that some years ago a tunnel was driven under the river bed for a short distance from the Arizona shore to provide filtered drinking water for the inmates of the Arizona penitentiary, which was about a quarter mile upstream from the location selected for the siphon. This tunnel is said to have extended about 50 ft. from the river bank and was excavated in material so tight that not enough water could be obtained for domestic uses. When openings were made in the roof to increase the amount the silt closed them up, and this method of obtaining water was abandoned.

The profile first selected for the tunnel is shown on Fig. 3 by the dotted lines. It was so situated that at no point did the tunnel approach nearer than 20 ft. to the surface of the sandstone, the shafts on either side being 130 ft. in depth. When it became necessary to use compressed air the elevation of the tunnel was raised to reduce the working pressures. The full lines show the work as built.

The tunnel is designed to serve 55,200 acres in the Yuma Valley and 40,000 acres on the Yuma Mesa—a total of 95,200 acres. The maximum use of water was assumed in July, when experience shows that one second-foot continuous flow will serve about 68 acres of diversified crops. The capacity of the siphon should then be 95,200 ÷ 68 = 1400 sec.ft. The structure is designed to pass this quantity with a loss of head of about 2 ft. After considerable study of various formulas a circular section 14 ft. in diameter was fixed upon as approximating the desired conditions. The thickness of the shell of the tube is 24 in.

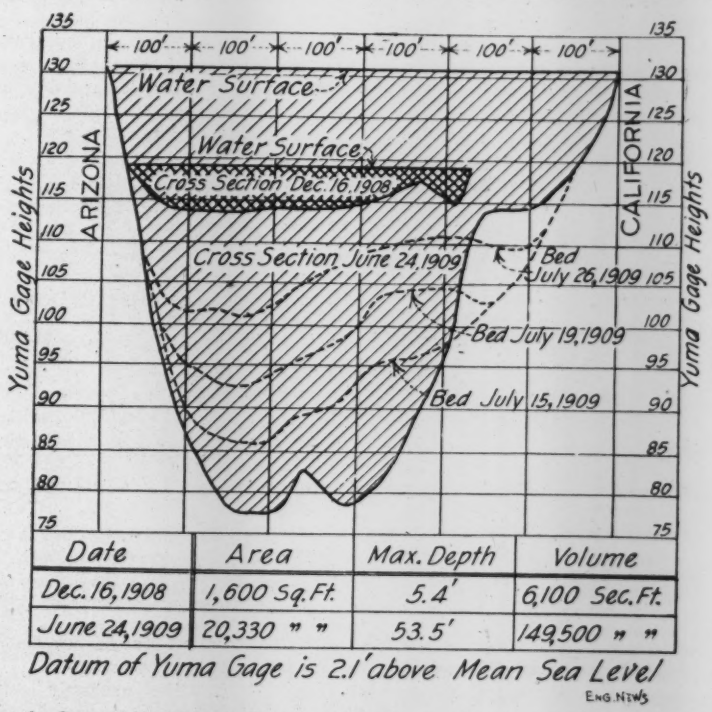


FIG. 2. CROSS-SECTIONS OF COLORADO RIVER, AT YUMA, FOR VARIOUS STAGES OF A FLOOD

(Showing deepening of bed as water rises, and filling of bed as water falls.)

DESIGN OF SHAFTS—Before deciding on this feature, a careful examination was made of various shafts and bridge

piers which have been successfully constructed in the United States, in Canada, and in Europe by the open-caisson method. Some characteristics of the principal ones examined are given in Table I herewith.

These structures vary in diameter from 10 ft. to 75 ft. and in depth from 65 to 100 ft. The piers of the Poughkeepsie bridge were also sunk by this process to a depth of 120 ft. or more. The method consists in building the caisson as an annular ring, generally circular, but often octagonal, with walls sufficiently thick to resist the pressures encountered. When the shaft is on shore a pit to meet the outside dimensions of the caisson is first dug to a depth of 12 or 15 ft. In the bottom of this pit the cutting-edge is assembled and upon it the caisson is built to a few feet above the ground surface. Excavation inside the caisson is then conducted in such a manner that the structure gradually sinks into position and its walls are built up from time to time as their tops approach the surface of the ground. The shafts and piers shown in the above table have been constructed in this way, generally with little difficulty. It appears from the eight structures listed that concrete has played an important part in all of them except at the Siphon de Clichy, where cast iron was used. Excluding the Wilkes-Barre shaft, which is built of reinforced concrete, the masonry has in all cases been protected by metal or timber attached to the cutting-edge, the masonry being used as a filling between these protective skins. There appears no good reason, however, why these protective skins could not have been omitted in nearly all cases, especially where the structure was surrounded by earth.

H. L. Wiley, who had charge of the piers sunk at Fort William, Ontario, says, on page 133 of Vol. LXII, *Trans.* Am. Soc. C. E.:

> In many cases an all-concrete caisson could be used to great advantage. The cutting-edge of the caisson could be protected and strengthened with a sheathing of steel plate for 5 or 6 ft. A channel riveted to the inside lower edge would hold the first section in shape and offer support for the concrete forms at the cutting-edge. This channel would be subject to the heaviest duty, and with the plate would prevent serious damage if boulders or timber should be encountered.
>
> The steel shell of an open caisson is often of no actual value except as a form for the concrete, and in cases where it might easily be eliminated it would seem to be an unnecessary expense.

Experience with the other caissons described indicated that a concrete structure with no reinforcement except at the cutting-edge and some few rods at the junction of successive days' work would answer all requirements, especially for such locations as Yuma, where silt and sand with practically no obstructions were anticipated, and so the shafts of the Colorado Siphon are of this type.

SIZE OF SHAFT—Under ordinary conditions the diameter of the shafts should be the same as that of the tunnel, in this case 14 ft. The uncertain conditions made it necessary to provide for all contingencies, and there was a possible chance that a shield might have to be introduced. The Arizona shaft, from which it was proposed to drive most of the tunnel, was given a finished diameter of 20 ft., and the California shaft 14 ft.

THICKNESS OF SHAFT WALLS—An examination of the details of the various caissons listed above shows that the concrete walls have in general had a thickness of about 5 ft. This thickness has demonstrated its ability to resist the strains due to sinking and give a practically watertight structure when in place. The caisson should, if possible, have sufficient weight to overcome the skin friction encountered in sinking, as the expense and annoyance of adding weight for this purpose will be eliminated. The amount of this skin friction varies widely with different materials: At Fort William it was from 275 to 300 lb. per sq.ft.; at the Blackwall tunnel, 560 lb. per sq.ft.; in the design of piers for the Manhaftan bridge it was assumed in sand as 500 lb. per sq.ft. (ENG. NEWS, Oct. 14, 1909, p. 402). At the piers on the Red River in Louisiana its intensity is not given, but the statement is made

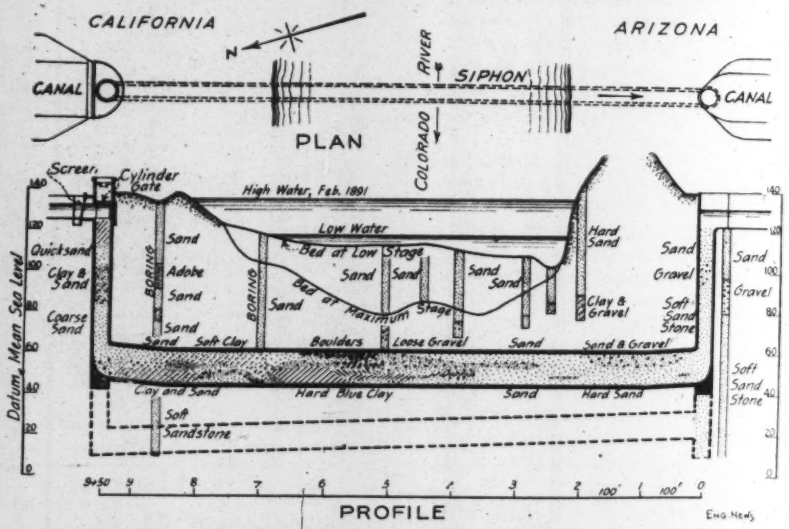


FIG. 3. PLAN AND PROFILE OF COLORADO RIVER SIPHON, AT YUMA, ARIZ.

TABLE I.—PRINCIPAL DETAILS OF SOME CONCRETE CAISSONS SUNK BY OPEN METHOD

Location	Depth	Diameter	Material	Kind of caisson	Thickness of walls of caisson	Skin friction	Time sinking	Concrete proportion
Wilkes-Barre, Penn. Coal Mine Shaft, Eng. News Sept. 24, 1908	80 ft.	59'6" by 28'0"	Sand, gravel, and quicksand	Reinforced concrete	7 ft. on sides 5 ft. on ends 2 ft. 8 ins. on top		1.5 ft. in 10 hrs.	1:2:4
Ft. William, Ont., Bridge Piers, Transactions LXII	60 ft.	31 ft.	20 ft. water; 40 ft. sand, gravel and clay	Concrete inside steel shell	6 ft. 6 ins.	275 to 300 lbs. per sq. ft.	5 mos.	1:3:5
Red River, La., Bridge Piers, Transactions LXII	90 ft.	37 ft.	25 ft. water; 65 ft. sand gravel, and clay	Timber lined with concrete	5 ft. 0 ins.	Sunk by own weight	4 ft. in 10 hrs.	1:2:4
Omaha Bridge, Pivot Pier, Transactions LXII	[illegible] ft.			Double steel cylinder with concrete				
Glasgow Harbor, Tunnel Shaft, Copperthwaite, p. 153	75 ft.	76 ft.	Sand clay	Double C. I. shell, with concrete	4 ft. 0 ins.		8 ft. per month	
Blackwall Tunnel, London, Copperthwaite, p. 181	77 ft.	48 ft.	Clay, sand	Double steel cylinder with concrete	5 ft. 0 ins.	560 lbs. per sq. ft. at bottom	1 ft. in 24 hrs.	1:6
Siphon de Clichy, Paris, Copperthwaite, p. 157	77 ft.	10 ft.	Sand, grav-marl.	Cast-iron segments			10 ft. per mo.	
Greenwich Tunnel, London,* Copperthwaite, p. 232	65 ft.	35 ft.	Clay, sand	Double steel cylinder with concrete	4 ft. 0 ins.			1:6

*24 ft. open; remainder in compressed air.

that the caisson with walls of concrete 5 ft. thick sank of its own weight. Mr. Wiley, in his article on the Fort William bridge piers, says: "A wall thickness of 5 or 6 ft. gives weight enough to overcome any friction which may develop ordinarily." Provision was made to give our shafts a wall thickness of 5 ft. so there would be approximately 5 cu.ft. of concrete opposed to each square foot of outer skin, and assuming the weight of masonry at 140 lb. we would have 700 lb. of caisson for each square foot of skin. This would overcome any of the frictions given above. It was believed, however, upon considering the materials through which the structure would pass,

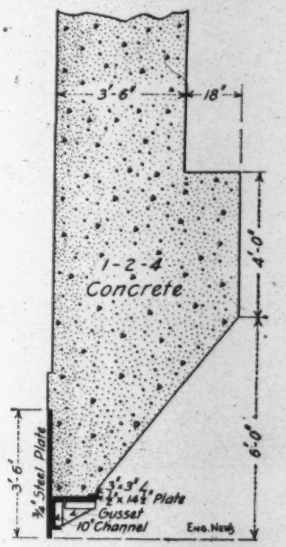


FIG. 4. SECTION AT CUTTING-EDGE, ARIZONA SHAFT OF YUMA SIPHON

that this provision was excessive, and at a point 10 ft. above the cutting-edge an offset of 18 in. was made in the inside skin, reducing the wall thickness to 3½ ft.; this increased the diameter of the Arizona shaft to 23 ft., and the diameter of the California shaft to 17 ft. This offset would provide a favorable support for an air-deck in case it became necessary to resort to the pneumatic process for sinking, and if additional weight was required it could readily be placed upon this shelf.

SINKING THE ARIZONA SHAFT

In the latter part of November, 1909, preliminary operations for sinking the Arizona shaft were inaugurated. The ground in this vicinity was leveled to an elevation of about 147 above mean sea-level. A derrick was installed, and a pit meeting the outside dimensions of the shaft was excavated to a depth of 10 ft. In the bottom of the pit the cutting-edge (Fig. 4) was assembled. This cutting-edge consisted of a ¾-in. steel plate 3½ ft. high, to which was riveted along the lower edge a 10-in. channel, this channel being reinforced by gussets about 2 ft. apart on the circumference. Supported by the channel and gussets was a ½-in. plate upon which the concrete of the caisson rested. Six feet above this ½-in. plate the concrete had battered inwardly to a thickness of 5 ft This thickness continued for 4 ft. more, or to 10 ft. above the cutting-edge, where the offset of 18 in. in the interior skin (above referred to) was made. From this offset to the top, the walls of the caisson were 3 ft. 6 in. thick. The bottom 10 ft. of the cutting-edge was heavily reinforced with twisted steel rods. There was also introduced a system of 2-in. pipes arranged to discharge on the outside of the caisson near the bottom of the cutting-edge in case material was encountered where the injection of water at this point under high pressures would assist in the sinking. There never was a time when this system of pipes was believed to be of value, and it was not used.

The bottom 10 ft. was constructed with wooden forms, but above this point steel forms were used. These were arranged to inclose an annular ring of the dimensions of the caisson walls, the total height of the forms being about 4 ft. When the concrete had hardened sufficiently the forms were slacked and raised to their next position with a derrick, then by bolts and nuts were tightened against the finished concrete and were ready to receive the next section of masonry. The cutting-edge was assembled with its bottom at elevation 136 ft. above sea level. The lower 10 ft. of the caisson was complete and sinking operations began on Dec. 27, 1909.

The material encountered to El. 120 was a perfectly dry, hard sand. This was handled by picks and shovels and removed from the caisson by derrick and bucket. The cutting-edge reached El. 120 on Jan. 7, 1910. At this point water was encountered in small quantities, continually increasing as the work went on. At first ordinary sinking pumps with capacities ranging from 150 to 300 gal. per min. were installed, but as the volume increased it was clear that the delays incident to pump repairs necessitated by the continual clogging from the sand and the eating out of the packings by the same agent would soon be a serious matter. The space occupied by the pumps was also a detriment to efficient operations. Finally this type of pump was abandoned and there was installed a vertical 6-in. Byron Jackson two-stage centrifugal pump belt-connected to a 65-hp. gas engine. This gas engine was later replaced by a 100-hp. steam engine. This pumping unit handled at times 1200 gal. per min. against a head of 100 ft.

Excavation still continued by pick and shovel, and the cutting-edge on Mar. 9 had arrived at El. 74. At this point, when the water was pumped down sufficiently for excavation to continue, the cutting-edge was under a hydrostatic head of about 46 ft., and from Mar. 2 to 9 repeated "blows" under the cutting-edge occurred, driving the men from the works. It was apparent that we had reached the limit of operations under such conditions, so it was decided to allow the caisson to flood and dredge the material. The derrick was arranged to handle a ½-yd. clamshell bucket, and on Mar. 14 the operation of this new equipment began. The material to be dredged was quite hard, and it was necessary to attach steel teeth to the bucket in order to penetrate it. Use was also made of a piledriver hammer weighing 2000 lb., to which steam-shovel teeth were attached. This hammer was used as a ram for breaking up the bottom in advance of dredging operations.

FIG. 5. AIR LOCK AT ARIZONA SHAFT ENTRANCE TO TUNNEL

On Mar. 16 the cutting-edge reached El. 69 and the excavation had been carried to a point 10 ft. below, but the caisson failed to penetrate. The clamshell, of course, could not reach to the outer edge of the caisson, so after the core had been dredged there still remained an annular ring of material that was sufficiently strong to support the caisson. The centrifugal pump was installed on the 16th and the water on the inside lowered a few feet. The caisson immediately began to settle, a penetration of 36 in. being accomplished in 1¼ hr. On Mar. 16, when the cutting edge was about El. 66 and the core was several feet below the cutting-edge, no penetration occurred. The pumps were started and the water inside lowered to a point about 15 ft. above the bottom of the caisson. A blow occurred, followed by an immediate penetration of about 6 in. This blow resulted in some disturbance of the surface immediately outside the caisson. On Mar. 20 the cutting edge had reached El. 65 and the excavation was 10 ft. below this point. Lowering

the water by pumps gave no penetration. On Mar. 23 the water inside was 7 to 8 ft. above the cutting-edge. No blow occurred and penetration was accomplished. Later on the same day, with the water 23 ft. above the cutting-edge, a blow occurred. On Mar. 28 the cutting-edge reached El. 58. Lowering the water inside a reasonable amount failed to produce any movement of the caisson and it was deemed inadvisable to lower the water further, because of the blows which usually followed.

Pipes were jetted down on the outside of the caisson to about 5 ft. below the cutting-edge. The excavation on the inside was carried about 10 ft. below the cutting-edge and dynamite was exploded in the outer holes to break up the annular ring which was preventing the movement. This method of operation was carried on until Apr. 6, when the cutting-edge had reached El. 55.5. It

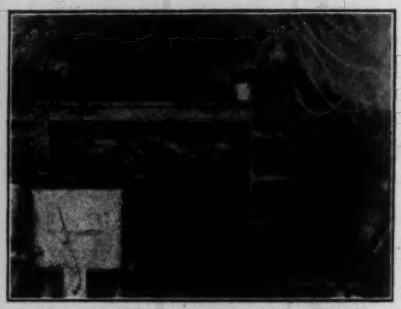


FIG. 6. ARCH HEADING WORK IN TUNNEL; SANDBAG BULKHEAD AT END OF INVERT EXCAVATION

was very difficult to place these outer holes properly, and the results from them were uncertain. On Apr. 6 blasting on the outside, assisted by light charges on the inside, gave a penetration of only ⅞ in., but did result in a heavy blow. It was then decided to change our method of operation and use divers.

In the plan adopted excavation was made by the clamshell bucket to a point 8 or 10 ft. below the cutting-edge. The pump was then used to lower the water to about 50 ft. above the cutting-edge, where there was no danger of a blow, and hold it there. A diving stage was suspended just above this water level and a diver went down to the bottom of the excavation. With a long auger holes were bored in the walls of the excavation about 5 ft. below the cutting-edge, extending to the line of the outer skin of the caisson or a little beyond. Two sticks of 40% dynamite were placed in each hole, connected by wires, and fired by a battery from the surface after the diver had come up. The divers began operations on Apr. 7, when the bottom of the cutting-edge was at El. 55.5.

On Apr. 28 the cutting-edge had reached El. 40 and the divers reported that all material had been shot free from below. The caisson refused to move, however, and as near as could be determined a skin friction of over 400 lb. per sq.ft. was holding it. Water jets were introduced around the outer skin to relieve this friction. By May 6 the cutting-edge had reached El. 31. The skin friction appeared to be almost entirely removed and a light charge of dynamite inside of the caisson produced a penetration of 3 ft. 3 in. immediately. On June 1 the caisson again refused to move and showed an apparent skin friction of about 460 lb. per sq.ft. Jetting was again resorted to and on June 5 the cutting-edge had arrived at its final position, El. 8.3 above mean sea-level. This was about 10 ft. below the grade selected for the tunnel, and the sinking operations were suspended.

The caisson when landed had a total weight of 2500 tons.

SINKING THE CALIFORNIA SHAFT

The methods employed in sinking the caisson at the California shaft followed substantially the same lines as those adopted on the Arizona side. The cutting-edge was of similar design, its diameter being 24 ft. The steel forms for concrete were the same, dimensions only being changed.

Preliminary work at this point began in January, 1910. The cutting-edge was assembled with its bottom at El. 118. Sinking operations began on Feb. 14 and continued in the dry by ordinary methods until Feb. 28, when the cutting-edge had reached El. 98.6. At this point water entered the excavation and the caisson was allowed to flood and the material inside was removed by means of a clamshell bucket. This mode of operating continued until Mar. 18, when the cutting-edge had arrived at El. 61. During this latter period the caisson became stuck on a number of occasions and it was necessary to jet pipes on the outside below the cutting-edge and shoot out the annular ring of material that was supporting the caisson. At times when the bottom appeared to be free the jets were used around the entire outer surface of the caisson to break the skin friction.

At El. 61 examination on the inside of the structure with a jet indicated that the caisson had reached a very hard material on the river side and that this material dipped toward the land at the rate of about 8 ft. in 24. This stratum prevented penetration on the river side of the structure, but the space between the cutting-edge and the hard material on the land side allowed the loose sand from above to enter the caisson as the work of dredging progressed. This caused some subsidence of the surface. Holes were jetted into the material on the high side and it was shot out of the way. Later it was necessary to use divers on the inside of the caisson as was done at the Arizona end. The structure was landed in its final position with the cutting-edge at El. 28 on Sept. 30. The caisson when landed weighed 1565 tons.

During the sinking of this shaft it was never feasible to assist the penetration by pumping down the water on the inside, as this was invariably followed by a blow. In fact, most of the time it was necessary to keep the water in the caisson higher than that outside to prevent these blows. At times the divers operated in 110 ft. of water. During the lat-

FIG. 7. VIEW IN COMPLETED TUNNEL OF YUMA SIPHON

ter period of sinking the caisson was loaded with about 320 tons of steel rails and cement.

SEALING THE SHAFTS

During the sinking of the caissons the hard material surrounding their lower portions was shattered for varying distances beyond the outer skin of the structures. This irregular annular space had become filled with fine sand and water which it was necessary to consolidate before a tunnel could be driven from the caisson, unless the pneumatic process was resorted to.

Foundations in coarse sand and gravel have been successfully constructed by injecting portland-cement grout under pressure, thereby filling the voids and creating a hard, compact substance. Willcocks in "Egyptian Irrigation" discusses operations of this nature at the Delta Barrage across the Nile below Cairo. Later a similar process was used at the subsidiary weirs below this dam, described in "Irrigation," by Hanbury Brown. In *Trans.* Am. Soc. C. E., Vol. LXVI, appears an article: "The Redemption of the Great Valley of California." In this paper and the discussion which

accompanied it reference is made to the feasibility of consolidating fine sand by the injection of cement grout. In ENG. NEWS, Vol. XXVII, p. 420, appears: "New Process of Dealing With Quicksand." This is an article describing a process patented by R. L. Harris for the creation of foundations in quicksand by the injection of portland-cement grout. A careful study was made of the data contained in the various articles and it was thought that under favorable conditions it might be possible to solidify the annular space surrounding this caisson by this method; after allowing the material sufficient time to harden a cut could be made through the walls and the tunnel driven into the indurated sand beyond without the aid of compressed air. The attempt was made at both shafts, but as the methods followed were the same in each case a description of the work at the Arizona shaft will be sufficient.

The caisson at this point, which had a circumference of about 100 ft., was surrounded with 4-in. pipes generally 10 ft. apart, except that in the area through which the tunnel would pass the distance between pipes was made about 4 ft. The pipes extended to the bottom of the cutting-edge, or about 130 ft. in depth, and were placed approximately 1 ft. beyond the outer skin of the caisson. The grout was mixed as follows: 2 sacks portland cement, weight 190 lb., volume 3100 cu.in.; 0.96 cu.ft. water. weight 60 lb., volume 1660 cu.in.

The total volume of the mixture was 4760 cu.in. and its weight 250 lb. Assuming that water weighs 62½ lb. per cu.ft., the mixture had a specific gravity of 1.45. The grout was prepared by hand in a mixing box from which it was delivered to an airtight receiver. The receiver was connected by 2-in. pipes with the wells to be grouted, and also had a connection with a small air compressor capable of maintaining from 150 to 250 lb. per sq.in. The well was fitted with a ¼-in. telltale pipe extending to within 5 ft. of the bottom and passing through the well cap at the surface. This telltale pipe was provided with a small petcock, by opening which when the well was under pressure the ejection of air or water would show whether the grout was passing out of the well. The pipe under pressure was raised during the operations by screw-jacks, and it was proposed to gradually lift these pipes as the grout was injected until the material surrounding the caisson had been consolidated to a point well above the proposed tunnel roof.

About 7 cu.ft. of grout was injected into the first well in a few minutes. When the pressure was removed in order to prepare the apparatus to receive the second batch, it was found that the sudden removal of this pressure had caused the light sand surrounding the bottom of the pipe, which was under a hydrostatic pressure of about 48 lb., to blow back and the pipe was filled to within a few feet of the surface. To obviate this difficulty an independent connection was made from the air receiver to the well, and by a proper arrangement of valves the well was kept under continuous pressure. After this arrangement was in force we had little trouble with sand coming up in the pipe. It was found, however, that the amount of grout which could be injected in the various pipes, and in the same pipe at different elevations, varied greatly. At times it was impossible to inject any appreciable amount before the grout had set, and under such conditions it was necessary to remove the cap from the well, drill out the partially solidified grout, and jack the pipe up until the bottom was again free, when the grouting operations were continued. In some cases the pressure was sufficient to force the grout to the surface around the outside of the pipe, but could not cause any impregnation of the material.

The grouting was carried on until the bottoms of all the pipes had been jacked up to a point about 20 ft. above the tunnel roof, except in two or three cases where the tendency of the grout to come up on the outside of the pipes froze them in position and it was impossible to move them.

When the grouting operations were completed a concrete plug 6 ft. in thickness was placed under water in the bottom of the caisson, a bottom-dump bucket being used for this purpose. The placing of the plug was a continuous operation. The plug was given one week to set and the caisson then pumped out. Holes in which 2-in. pipes were inserted were drilled through the walls of the caisson on the line of the proposed tunnel, and it was found that the consolidation expected had not been accomplished. While there were small areas in which the grout appeared to have been effective, there were other areas where it apparently had no effect at all. Pressure gages applied to these pipes showed in some cases the full hydrostatic head and in other instances about half of that amount.

It was thought by driving 2-in. pipes through the walls of the caisson, distributed over the area of the tunnel about 2 ft. apart, and injecting grout through them at close range, better results could be accomplished. This method was given a fair trial, but the results obtained were no more satisfactory than when grouting from the surface. After this second series of pipes had been grouted, openings made through the wall into the area through which the tunnel must pass still showed high pressures and considerable flows of sand. It was then concluded that the only feasible solution of the problem would be the pneumatic process.

In the writer's judgment, it is impracticable to so consolidate fine sand by the injection of grout that the resulting material will serve any useful purpose. While sufficient pressure may be used to cause penetration, the fluid will move at such low velocities that the grout will set up before any appreciable distance has been traveled. It is believed that success attained by this method will be confined to coarse materials.

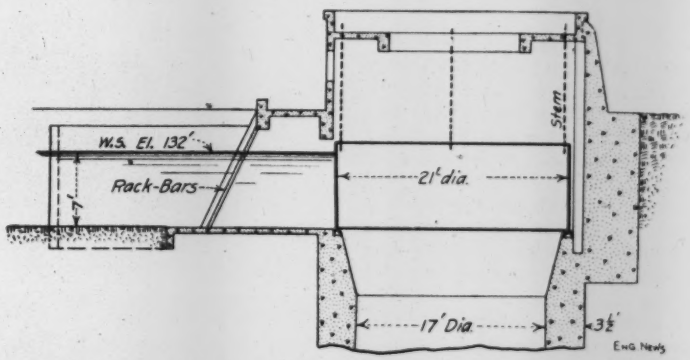


FIG. 8. SECTIONAL VIEW OF CYLINDER GATE AT CALIFORNIA SHAFT OF YUMA SIPHON

WORK UNDER COMPRESSED AIR

Pending arrangements for future operations, the work at both shafts was shut down on Feb. 21, 1911. All of the operations so far on the siphon had been carried on by day-labor under the direct supervision of the engineers of the Service. On account of the difficulties which had been encountered and which were to be anticipated, it appeared best to continue operations in the same manner.

As there was no pneumatic equipment of the kind required in the West, or men skilled in its operation, it was decided to get tenders on plant from Eastern localities where this class of work had been carried on. It was also thought advisable to convene at Yuma a Consulting

Board, one member of which was to be an engineer of wide experience in pneumatic work. In this connection the Service was very fortunate in securing the advice of Silas H. Woodard, M. Am. Soc. C. E.

The board, consisting of Mr. Woodard, Louis C. Hill, M. Am. Soc. C. E., Supervising Engineer in the Reclamation Service, and the writer, convened at Yuma on Apr. 10, 1911. All the data at hand were carefully examined and it was decided that the most feasible method of sealing the shafts to the sandstone would be by the pneumatic process. It was considered that once the seal was accomplished it might still be possible to drive the balance of the tunnel in normal air, but on account of the uncertainty of this it was believed best to install a sufficient pneumatic plant to drive the entire tunnel by this method if the conditions required. The board also examined the tenders which had been received for pneumatic equipment. These were of three classes: for new equipment, for second-hand equipment, and for the rental of plant accompanied by skilled compressed-air workmen. Because of the comparatively small amount of tunnel to be driven and the high cost of new plant, a further consideration of these tenders was rejected. The choice appeared to lie between the purchase of second-hand plant and the rental of a plant accompanied by skilled men. As there were no men in this vicinity trained in compressed-air operations, it was deemed highly desirable to take advantage of an existing organization. The tender of Charles A. Haskin, of Boston, was accepted. In addition to equipment, Mr. Haskin provided at his expense a superintendent of long experience, also 25 seasoned compressed-air workmen who were carried on the pay rolls of the Service. The equipment consisted of three compressors, a vertical lock, a horizontal lock, a medical lock, necessary tubing to connect the vertical lock to the air-deck, and miscellaneous accessories common to these operations. The compressors had a combined capacity of 4000 cu.ft. of free air per minute compressed to 40 lb. pressure.

The plant arrived at Yuma on May 26, 1911, and its erection was immediately commenced. An air-deck of timber 24 in. thick was constructed in the shaft at El. 83.1. The vertical lock was set at the surface and 4-ft. tubing extended from the lock through the air-deck. The air pressure was first put on June 19. The shaft being previously filled with sand to El. 42, a concrete plug 5 ft. thick, heavily reinforced with steel, was placed with its top at El. 47. The tunnel was then driven for 50 ft. from the Arizona shaft, the end bulkheaded, and the air taken off. In this 50 ft. of completed tunnel the horizontal lock, which was 6 ft. in diameter and 22 ft. in length, was installed. Cages were put in the shaft, and on Oct. 12, 1911, the driving of the remainder of the tunnel under compressed air was inaugurated.

By this time it was apparent that it would be necessary to carry pressure for the entire length of the tunnel, and preparations were made to that end.

The difference in pressure required for the upper half and for the lower half of the tunnel made it advisable to drive the upper portion first. The general program was to drive about 10 ft. of the upper half, which was supported by segmental plates of No. 11 steel. The plates were 1 ft. by 3 ft., measured on the circumference of the arch. Around the edges of these plates 2-in. angles were riveted, through which were punched holes for ¾-in. bolts. These holes were accurately spaced a uniform distance apart, so that it was possible to break joints in every ring of plates. While the excavation was being made the plates were held in position by trench braces resting on the bench. The main use of the plates was to prevent the escape of air. After 10 or 12 ft. of the upper portion of the drift had been driven, concrete ribs with wooden lagging were put in position and the masonry placed. By this method 50 or 60 ft. of arch was completed. The pressure was then raised to meet the conditions of the invert and the bottom half of the tunnel constructed in 10-ft. lengths. In general, about 21 ft. of the top of the arch was protected by plates, and only in rare instances was it necessary to extend the plates to the spring line. In a few places temporary supports were required on the invert for a short distance below the springing line, but in the main no invert supports were necessary. After a length of invert had been completed to within about 2 ft. of the arch, a bulkhead of sandbags was built at the end to prevent water from coming in on the work while the arch was being driven under lower pressure.

Fig. 5 is a view looking into the horizontal lock at the bottom of the Arizona shaft. Fig. 6 is an interior view of the tunnel showing the sandbag bulkhead at the end of the invert and work on the arch in progress. Fig. 7 is a view [illegible] completed tunnel.

The last masonry under air pr[illegible]re, making the junction with the Cal[illegible]nia shaft, was placed on May 14, 191[illegible] On May 21 the air pressure was redu[illegible]d to 5 lb., a few spurting leaks were n[illegible]ked, and the pressure again raised. [illegible]ese leaks were calked and on June 2, 1912, the pressure was entirely removed. The leakage at that time was so slight that further air pressure was deemed unnecessary and the work of dismantling the plant began. On June 4 the California shaft was unwatered and the work of cutting through the walls of this shaft from the tunnel was carried on under normal air.

The air pressure has in general been between 27 and 32 lb. above atmospheric. While there were a few cases of caisson disease, serious ones were extremely rare. There was one fatality. Partial paralysis from the air, combined with other causes, resulted in the death of one man in April.

The siphon was put in service on June 29, 1912.

FIG. 9. ARIZONA MAIN CANAL, 1 MILE BELOW YUMA SIPHON

GENERAL REMARKS

The entrance of water to the Siphon is controlled by a cylinder gate installed at the top of the California shaft, shown in Fig. 8. This consists of a steel cylinder suspended from three points by threaded stems connected on top of the shaft by gears to an electric motor. The gate is arranged to slide freely in its guides, and the water approaches from all sides. The gate is protected by rack bars a short distance above, as shown in the sketch. This gate was designed by F. Teichman, M. Am. Soc. C. E., Designing Engineer U. S. Reclamation Service. (See also view on first page.)

About 2000 ft. above the California shaft, waste gates have been constructed for turning the entire flow of the canal into the Colorado River as occasion demands, or for wasting at this point that portion of the flow which is not required to pass the Siphon. Waste gates were provided at this point because it would be difficult to properly control the Siphon from the diversion at Laguna

Dam 15 miles above. A further reason was that a power plant is to be installed about three miles above the Siphon, and a wasteway as built would allow a continuous load on the wheels, which would be subject to a varying flow if the control of the canal was made at Laguna Dam.

The Siphon discharges into a canal of 80 ft. bottom width and 7 ft. water depth, which for a short distance is in a cut 25 ft. deep. A typical structure on this Project is shown by Fig. 9, being a view of a bridge across the main canal about one mile below the Siphon.

Frederick H. Newell, M. Am. Soc. C. E., is Director of the Reclamation Service, and Arthur P. Davis, M. Am. Soc. C. E., is Chief Engineer. The Yuma Project is in the Southern District, of which Louis C. Hill, M. Am. Soc. C. E., is Supervising Engineer.

During the sinking of the caissons Richard P. Marable was Superintendent at the Arizona Shaft and Frank W. Hall at the California Shaft. The grouting operations were conducted under the direction of R. M. Priest, Assistant Engineer. On the pneumatic work George Schobinger, Assoc. M. Am. Soc. C. E., was Assistant Engineer, and Edwin C. Hayden, Assoc. M. Am. Soc. C. E., was Superintendent.

The writer has had responsible charge of the designing and execution of all work on the Project since October, 1906.

Development of the Seattle Municipal Electricity Supply System

The 1911 report of the Lighting Department of the City of Seattle, Wash., recently printed, contains a very complete history of the project, with an account of the conditions under which it has operated and a summary of the books.

Agitation for this plant came to a head in 1902 when the Board of Public Works, at the direction of the City Council, submitted a report outlining plans for a municipal electric-supply system. Construction work was begun in April, 1902, and in October, 1904, lights were turned on at the generating plant. Commercial service began with 1905.

The source of power for the municipal system is Cedar Lake, in the Cascade Mountains, about 43 miles southeast of Seattle.* The lake discharges into Cedar River, a swift mountain stream, which falls 600 ft. in 3½ miles. There is a rock-filled timber dam near Cedar Lake and long wood-stave pipe lines leading to the generating station at the lower level.

There is at present a maximum machinery capacity of 18,000 hp., but to secure more than about 11,000 hp. an additional pipe line and more storage will be necessary. For this, a concrete dam is being constructed on Cedar River, 2¼ miles below the lake. This will raise the level 58 ft. above the present dam and increase the size of the lake. The total power available after this work is done will be 20,000 hp., continuous load. As the plant-load factor is about 40%, a peak load capacity of 50,000 hp. can be installed. Cedar River is also the source of the city water supply and the new dam will increase the capacity of the waterworks as well as of the power system.

The municipal plant competes with a local electric company which is a consolidation of two smaller companies. While there is competition, yet there seems to be a friendly feeling between the utilities. The city system has carried from 1000 to 5000 hp. of the private company's load on 38 days, in all, when there were fuel shortages, and the city plant was given a considerable part of its load on 22 days during a recent shutdown from floods.†

Previous to the definite inception of the municipal-system plan, in 1902, the private electric company had charged 20c. per kw.-hr. for residence supply, but at that time the rate was reduced to 12c. The city supply was started in 1905 with a residence rate of 8½c. per kw.-hr. for the first 20 kw.-hr., 7½c. for the next 20, 6½c. for the third 20, and 4½c. for all excess. This schedule was followed by a corresponding reduction in the central-station company rates. In 1911 the rates were further reduced, so that for residence lighting the price for the first 24 months after meter installation is 7c. per kw.-hr. up to a load of 60 kw.-hr. per month, and 4c. in excess. After 24 months these figures become 6c. and 4c. respectively. For business lighting the rate varies from 8c. down to 1½c., according to the connected load and the actual use. For power supply below I hp. connected load, the business rates apply; for larger motors the rates vary from 7c. to 0.6c. The minimum charge for residence service is 75c. per month, for business service $1 per month, and for power supply $1 per hp. per month. The street lighting is supplied under an allowance from the general city fund of $54 per year for 400-watt arc and incandescent lamps, or at about 4c. per kw.-hr.; incandescent cluster lighting is supplied at $42 per year per 200-watt cluster, or at about 6c. per kw.-hr.

The growth of the service is shown in the accompanying table which is compiled and condensed from the very full statements printed in the 1911 report. In connection with this table, it should be stated that interest on the municipal light-and-power bonds is raised by a tax levy and paid from a special fund known as the interest fund. On the annual budget the interest fund is charged with the amount of this interest and against this charge on the credit side of the budget is entered an item equal to the total annual interest on the light-and-power bonds, thus making a reduction in the total tax levy equal to the total plant-bond interest charges. The general fund is then reimbursed from the light fund (earnings). Any unpaid balance of interest is shown under the head of liabilities, being due the general fund.

Depreciation is based on a schedule of the life of the different parts of the plant, drawn up by the engineers. The same percentage is taken each year from the diminished balance and this depreciation is charged against the revenues, setting up an account called "reserves for depreciation" against this charge. Plant maintenance is offset against this depreciation taking the same from the total depreciation fund. No actual fund for depreciation has been created, but the amount has been reinvested in extensions. The yearly surplus shown has also been used in extensions.

*See Engineering News, June 17, 1909.
†See Engineering News, Dec. 14, 1911.

FINANCIAL OPERATIONS OF THE SEATTLE MUNICIPAL ELECTRICITY SUPPLY SYSTEM

	Expense					Revenue			Assets			Liabilities		
	Generating	Distributing	General	Fixed Charges[1]	Total	Business and Residence[2]	Power	Street Lights	Total	Accounts[3]	Property[4]	Depreciation Reserve[4]	Accounts[5]	Bonds	Annual Surplus[6]
1905	$11,601	$9,352	$2,706	$40,688	$64,347	$2,188	$266	$43,016	$45,470	$121,657	$734,935	$.....	35,569	840,000	—$18,877
1906	18,634	28,391	9,669	71,631	128,325	46,235	4,917	66,148	117,300	68,895	899,620	39,506	158,417	840,000	—11,025
1907	25,272	48,904	19,424	81,106	174,707	122,311	9,951	66,531	198,793[3]	149,916	1,269,959	79,012	285,692	1,040,000	—5,816[6]
1908	24,047	75,536	37,174	154,550	291,307	205,643	37,952	74,245	317,840	38,964	1,875,626	115,328	453,872	1,440,000	+20,718
1909	33,394	84,942	48,634	201,958	368,928	276,661	102,880	88,846	468,387	268,940	2,143,281	308,391	372,761	1,940,000	+99,459
1910	39,146	100,323	81,532	274,087	495,088	359,004	126,016	113,495	598,515	215,371	2,737,975	503,928	709,918	2,140,000	+103,427
1911	71,360	117,860	138,190	246,539	573,949	461,361	97,189	168,833	727,383	403,918	3,210,795	664,797	1,221,279	2,240,000	+153,434

[1] Includes depreciation and bond interest.
[2] Includes receipts at business, residence and flat rates and miscellaneous items.
[3] Includes receivables, fund balances available, etc.
[4] Depreciation reserve is reinvested in extensions and so included in property assets. Not a separate asset here.
[5] Includes overdrafts on appropriations, interest paid from taxes, loans from other funds, tax apportionments, accumulated surplus, etc.
[6] A deficit for 1905-6-7. There was a net yearly surplus in 1907 of $24,086 which was applied to the accumulated deficit.

Cab Signal and Dispatching System

A combined train-dispatching and signaling system for electric railways, in which cab signals are used and which may be supplemented by an automatic stop, is now in use on one railway in this country and another in Canada. It has been in service for about three years on the Toronto & York Radial Ry. The more recent installation, which was put in operation in June, is on the line of the Indianapolis & Cincinnati Traction Co., and was installed under the direction of C. L. Henry, President and General Manager. It covers about 18 miles of single track (between Indianapolis and Shelbyville, Ind.), and includes ten sidings, which are of the stub or dead-end type. The length of blocks (extending between sidings) averages two miles. Regular trains are run at intervals of 60 min., and at speeds as high as 60 m.p.h. The system is the invention of Paul J. Simmen, and is handled by Northey-Plummer (Limited), of Indianapolis, Ind., and Toronto, Canada.

Signaling systems which give indications in the engine cabs (instead of at posts along the track) have been tried experimentally by steam railways here and abroad, but we describe herewith a system of this kind which is in regular service on two electric interurban railways, and by which the indication in the motorman's cab is continuous, changing from "clear" to "danger" (and the reverse) according to conditions. It is a feature of a combined train-dispatching and block-signaling system, which is said to be fully as effective and economical as any automatic signaling system. A special auxiliary feature (which is not included in the installation described) is an automatic stop which can not only stop a car or train which over-runs a danger signal but can be applied to regulate the speed of trains at curves or other points where slow speed is required. The signal system and automatic-stop apparatus are of particular interest at the present time, as the necessity of operating electric interurban railways under the protection of the block system is being generally recognized, and block signals are being adopted extensively on such roads.

Dispatching System

The movements of cars or trains are governed by orders issued by the dispatcher, but an automatic train record at the dispatcher's table shows a continuous record of all the movements on the line. This record is made on a traveling sheet of paper, divided transversely for time intervals and longitudinally for the several block sections or stations. A new sheet is placed in the machine each day. By means of a clockwork train the paper is fed across the table, or across a frame above the table (as shown in Fig 1). As it is fed forward the paper passes under a frame fitted with perforating needles, which move vertically and are operated by electricity. Two needles are provided for each block, to record movements in both directions. For single-track operation, the blocks extend usually between passing sidings.

When a car enters a block the needle corresponding to that block commences to perforate the traveling sheet of paper, and continues to do this until the car passes out of the block. This shows graphically the position of the car and whether it is running according to schedule. A time scale of 3 in. to the hour is considered sufficient for service up to 30 min. headway, but for heavier traffic a scale of 6 in. to the hour is recommended. A width of 14 in. will suffice for the record sheet for an average single-track division 100 miles long, the margin affording room for any notes by the dispatcher.

At the dispatcher's table is a set of control switches, each controlling the circuit for a particular siding. When the switch lever is vertical it causes a "stop" signal to be given to a car approaching that siding in either direction; when moved to the right or left it causes a clear signal to be given to cars moving in one or the other direction, respectively. The connections are so interlocked that cars in opposite directions cannot be given clear signals for the same block section. At each siding the car is automatically put in telephone connection with the dispatcher, the telephone being in the cab (instead of on a post or in a cabin) so that the motorman is not required to leave his car.

A typical wiring diagram for both the recording and signaling systems is shown in Fig. 2, the arrangement being for a line with double-end or through sidings and four contact rails at each siding. These contact rails or "third rails" are steel angles or bars, carried on insulating supports, and having the ends inclined to engage easily with the hinged or telescopic contact shoes on the car trucks. For high speeds the rails are 70 to 80 ft. long, those on the Indianapolis line being 72 ft. long. Ordinarily each siding will have four of these contact rails; a "home" rail opposite each switch and a "distant" rail 2000 ft. beyond. With stub sidings, as on the Indianapolis line, only three contact rails are used; one "home," with a "distant" 2000 ft. in each direction. Normally there is an open circuit between the contact rail and the track; but when a car shoe engages the rail this circuit is closed, and the circuit then is through the rail and goes to the apparatus in the cab and thence to a ground return on the truck and track rails. Fig. 3 shows the two "home" contact rails at one end of a double-end siding.

Referring to Fig. 2, the "home" contact rails at siding No. 6 are connected by wire *E* to the dispatcher's office, where they are connected to one side of the dispatcher's switch *D*. From the other side of this switch a wire *B* connects through the winding of the main relay *C* to the pole of the storage battery *A*, the other pole of the battery being in turn connected to ground at track rails *K*. The instant the contact shoe of a car makes contact with a contact rail, a circuit is established as follows: from storage battery *A* in the dispatcher's office, through wire *B*, main relay *C*, dispatcher's switch *D*, wire *E*, third rail *F*, contact shoe *G*, signal relay on cab *I*, ground on truck *J*, and track rail *K*, back to dispatcher's battery *A*.

The closing of this main circuit energizes the main relay *C*, which in turn closes a local circuit through the perforating magnet *N* as follows: from storage battery *O* through wire *M*, perforating magnet *N*, armature of main relay *L*, to negative side of storage battery *O*. The perforating magnet *N*, being energized, acts on its armature *Q*, which forces the perforating needle upward, thus perforating the dispatcher's record sheet and recording upon it the time and place of a train passing siding No. 6. The perforating circuits shown provide only for a single punch upon the recording sheet when a train is passing a contact rail. To give continuous perforation as long as a train is in the block, a polarized relay is used. This is not shown on the cut.

Signaling System

No outdoor signals are used, but indications are given directly to the motorman by means of lights in the cab. The connections are effected by the intermittent "third rails" or contact rails at the sidings, which engage with shoes on the cars. If the rail is electrically energized, a car passing it (and entering the block)

will receive a "clear" signal; otherwise, a "stop" signal will be displayed. The former signal will continue while the car is in the block, and the latter will continue until it is changed to clear by the car passing a contact rail which is energized. One or the other signal is always displayed, so that the indication is positive. Should two cars approach a siding from opposite directions (each having a clear signal through its block), each car would receive a "stop" signal when within 2000 ft. of the siding, the cars being then 4000 ft. apart. The indications are given by two small lamps (green and red) arranged separately or in a box at the right of the motorman, as shown in Fig. 4. At one side of the cab is the telephone for communication with the dispatcher's office.

The cab-signal circuit is shown in Fig. 5. Normally, the shoe *G* hangs down and its arm *R* is in contact with the stop *S*. When the shoe is raised by the contact rail *F* (as shown), the arm is thrown back from the stop. On the Indianapolis line, however, the stop moves vertically, telescoping into a chamber above it. If the contact rail is energized, a circuit is established through the shoe and the wire *H* to the signal relay *I*, then to earth at *J* and track *K*, and thence completing the circuit through the dispatcher's office, as described. With this circuit energized, the signal relay *I* will be energized and the front contact of its armature *T* will close the circuit through the green or clear-signal lamp as well as the other circuit to front stop *S*.

As soon as the shoe leaves the contact rail, the spring *U* forces the contact arm *R* against the front stop *S*. This closes a local holding circuit as follows: through the shoe *G*, wire *H*, relay *I*, storage battery *W*, front contact of armature *T* to the front stop *S* and the shoe *G*. In this manner the clear signal is continued throughout the entire block entered.

When a car passes a de-energized rail, a danger signal is displayed as follows: the shoe on the car being lifted by the contact rail, the contact arm *R* separates from the front stop *S*, thus breaking the local holding circuit and preventing the storage battery *W* from continuing to energize the signal relay *I*. Since the

FIG. 1. AUTOMATCIALLY RECORDING TIME SHEET OF THE SIMMEN DISPATCHING AND SIGNALING SYSTEM (INDIANAPOLIS & CINCINNATI TRACTION CO.)

energy from the dispatcher's circuit is also absent, the relay *I* becomes de-energized, its armature *T* drops, opening both front contacts of the clear signal and the local holding circuit and closing a circuit through the back contact and the red signal lamp. As the shoe leaves the contact rail, the contact *RS* is closed again; but the local holding circuit is now open at the front contact of the armature *T* and the danger signal is continued until the car shoe engages a contact rail which is energized.

A bell can be inserted in the danger circuit, so that both visible and audible danger signals will be given. This is shown in Fig. 5. The bell gives one tap each time a contact rail is passed, thus calling the attention of the motorman to a possible change of signals, as well as being a positive indication that the cab equipment is in proper operating condition.

The dispatcher's control of the signals may be explained from Fig. 2. As stated before, when a contact rail is energized, a clear signal is displayed in the car: when it is de-energized, a danger signal is displayed. Since the dispatcher's switch *D*, controlling the signals at siding No. 6, is closed, it is evident that the contact rails at this siding are energized, and therefore set for a clear signal. To display a danger or stop signal at the siding, the dispatcher simply opens switch *D*, thus depriving the contact rails of electrical energy from the storage battery *A*.

Interlocking arrangements on the dispatcher's switchboard prevent the dispatcher from giving a clear signal to a train entering a block until the signal at the opposite end of the block (for an opposing train) is set to indicate danger. Telephone communication is established over the same circuit as is used for signaling, the telephone apparatus being placed in shunt with the circuit and provided with condensers.

AUTOMATIC STOP

The device for automatically stopping a car which is entering a block in spite of a danger signal has not been applied either at Toronto (except experimentally) or at Indianapolis, but it forms a part of the complete system. The apparatus is intended to be condensed into the size and form of an ordinary speedometer for automobiles, and its control is based upon the cab-signal system described above. On a road equipped with this signal the automatic stop may be added without any alterations in the existing apparatus and with no greater cost than if it had been installed in the first place.

The operation of the stop is as follows: Whenever an engineman or motorman receives a danger signal at a "dis-

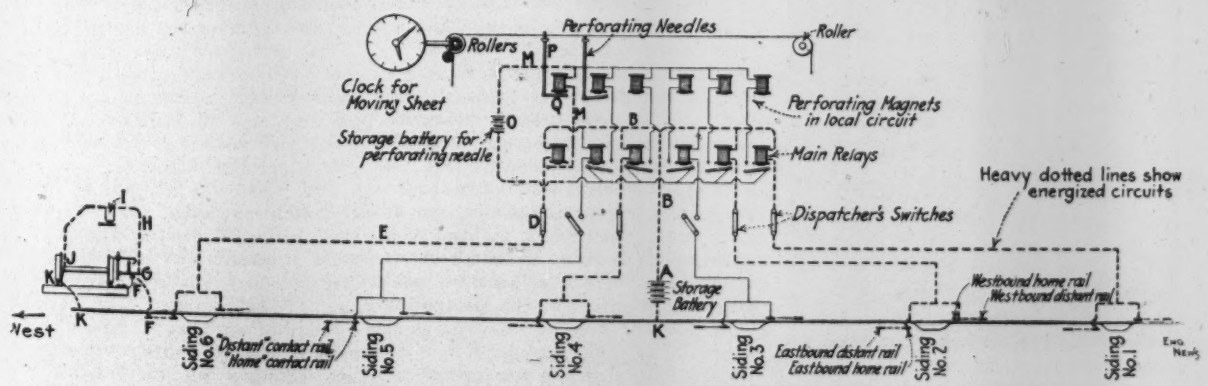


FIG. 2. WIRING DIAGRAM FOR RECORDING AND SIGNALING APPARATUS

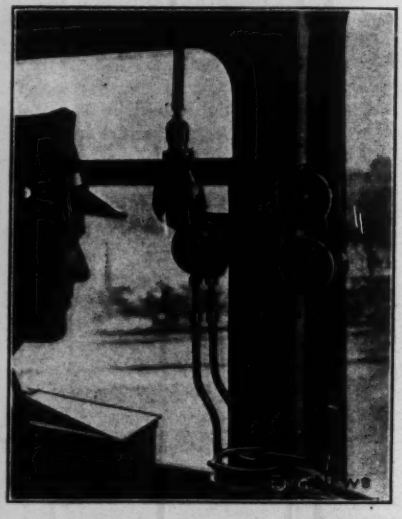
(Sidings Nos. 1, 2, 4 and 6 (with energized circuits) are set for clear signal. Sidings Nos. 3 and 5 (with non-energized circuits) are set for danger signal.
At siding No. 6 a train is passing an energized contact rail, receiving a clear signal and at the same time making a record on the train sheet in the dispatcher's office.)

tant" contact rail (as described above), the automatic-stop apparatus is set in motion, but this does not at once apply the air brakes. Should the car at any point between the "distant" and the "home" contact rails proceed at a greater speed than is permissible for a proper stop, a valve in the train pipe is opened automatically and the air brakes are applied, thus bringing the train automatically under control. The moment that the actual speed of the train has become less than the proper specified speed, the air valve is closed automatically and full control over the brakes is thus restored to the motorman.

FIG. 3. CONTACT RAILS FOR HOME SIGNALS
(Giving Signal Indications in the Cab)

Another feature of this stop is that a car may proceed past a "home" signal at danger and enter a new block, but under these conditions the motorman is compelled to keep within a predetermined speed under control throughout the entire block. He cannot resume unlimited speed until after passing a contact rail at which he receives a clear signal. Exceeding the speed limit in a block entered with a "danger" signal would result in the automatic application of the brakes.

A motorman approaching a curve, or about to pass over a stretch of track where a slow order is in force, may be compelled to reduce the speed to that suitable for the particular point; otherwise, the air brakes will be applied automatically. The only thing necessary to provide for such variable speed reductions is to erect a dead rail at a suitable distance from the curve. This sets the automatic stop in motion, and at the proper distance the shoe passes over a live rail, which again restores the automatic-stop apparatus to its normal position. The proper speed reduction for any point or curve would depend solely upon the distance between the dead and live rails mentioned. Further, the apparatus can be so arranged that a car or engine traveling backward must travel within a predetermined speed limit, and where a limit is imposed provision can be made for applying the air brakes automatically if such speed is exceeded.



SIGNALS IN MOTORMAN'S CAB; INDIANAPOLIS & CINCINNATI TRACTION CO.
(The two signal lamps illuminate the disks on the window post.)

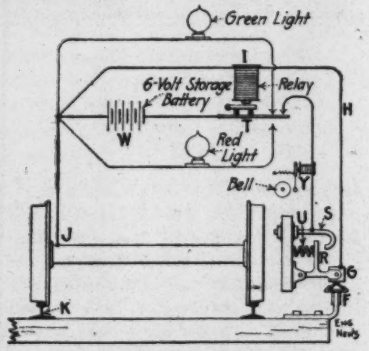


FIG. 5. DIAGRAM OF CIRCUIT FOR SIGNALS IN CAB OF ELECTRIC CAR

The broad features provided for in this stop are that the proper control of the speed is left entirely with the motorman (or engineman) so long as he does his duty; but when he fails to do this, the air brakes are applied automatically. The moment the speed of the train is again within proper limits, the control of the brakes is restored automatically to the motorman, whose only control over the stop apparatus is to run his train at a proper speed.

A Bureau of Farm Power in the Department of Agriculture is proposed in a bill introduced in the United States House of Representatives by Congressman Henry T. Rainey of Illinois. The bill provides for a chief of the bureau appointed by the President, at a salary of $6000 per annum. It would be the duty of the bureau to investigate and report upon all matters pertaining to methods of furnishing power on farms, and all labor saving machinery adapted to use on farms.

The Need of a Compression Test for Concrete

BY CLOYD M. CHAPMAN*

The materials used in engineering are usually tested to determine the degree to which they possess the qualities which especially fit them for the purpose to which they are to be put. Steel to be used in tension is tested in tension. Building stone and brick and blocks which are to be used in compression are tested in compression. They are seldom tested in tension because they are not used in tension as a rule, although the underside of a flat-stone lintel is often in tension. Other materials are tested for the qualities which they must possess to render proper service in the finished structure or product.

The testing of portland cement is nearly as old as the industry itself. The early tests were very crude and consisted chiefly in seeing if the cement would set up at all. The tests now commonly applied are designed to determine several of its important qualities. Chemical analysis shows whether it is a true portland; that is, whether the component elements are present in the proper proportion to constitute a cement of the desired qualities and to detect the presence of adulterants and detrimental compounds such, for instance, as an excessive percentage of sulphates. The soundness, or boiling test, shows whether the component elements are so combined with each other that the cement remains of constant volume after setting and does not swell or warp or crack. The specific gravity indicates chiefly the presence of adulterants. The sifting or fineness test shows what proportion of the cement is sufficiently subdivided to allow the particles to combine chemically with the water, and so crystallize and harden.

The setting-time test shows whether there will be time to make up the concrete or mortar on the job and get it into place before it has hardened, and whether it will then attain a proper hardness in a reasonable time thereafter.

The tensile-strength test of the neat cement at one, seven and 28 days shows the cohesion or chemical bond of the particles of cement with each other, and the rate at which this bond increases with age. Tensile-strength test of mortar made with one part of cement to three parts of sand shows the adhesion of these two materials and the bonding power possessed by the cement in the mixture.

These tests as they are usually applied to cement and to mixtures of cement and sand are all valuable and interesting and indicate qualities in a cement or mortar which it is desirable to know, but what do they tell of the one quality which must be possessed by cement or cement

*Engineer-in-Charge, Westinghouse, Church, Kerr & Co., New York City.

TABLE SHOWING RELATION OF COMPRESSIVE TO TENSILE STRENGTH OF CONCRETE

			Hand Made Specimens						Machine Made Specimens					
			28-day test			6-months test			28-day test			6-months test		
Mix. No.	Proportions Cement to Sand	Kind of Sand	Tension	Com.	Ratio Comp to Tension	Tension	Com.	Ratio Comp to Tension	Tension	Com.	Ratio Comp to Tension	Tension	Com.	Ratio Comp to Tension
1	3	3	4	5	6	7	8	9	10	11	12	13	14	15
		Cow Bay												
1	1-3	Under ¼ in.	254	2084	8.2	238	3113	13.1	441	3530	8.0	475	5054	10.6
2	1-3 (15-lb. lime)	Under ¼ in.	255	1469	5.8	226	2663	11.8	390	4031	10.2	380	6040	15.9
2	1-3 (15-lb. lime)	Under ¼ in.	232	1990	8.6	218	2860	13.1	351	3000	8.5	315	4900	15.6
4	1-4	Under ¼ in.	196	1296	6.6	206	2280	11.1	353	3122	8.8	405	4520	11.1
5	1-4 (15-lb. lime)	Under ¼ in.	180	1343	7.5	172	2460	14.3	337	2487	7.4	325	5013	15.4
6	1-4 (25-lb. lime)	Under ¼ in.	175	1528	8.7	...	3550	....	355	3205	9.0	338	5610	16.5
		Cow Bay												
7	1-3	Under ⅛ in.	293	1695	5.8	295	3190	10.8	377	4780	12.7	445	8380	18.8
8	1-4	Under ⅛ in.	228	1008	4.4	252	2537	10.1	266	2573	9.7	365	4720	12.9
9	1-3	Std. Ottawa	290	2406	8.3	352	4210	12.0	661	2936	4.5	...	5300	....
10	1-4	Std. Ottawa	222	1770	8.0	258	3230	12.5	601	3459	5.8	...	4800	....
11	1-5	Std. Ottawa	148	1740	11.7	219	2840	13.0	540	2949	5.5	...	5030	....
12	1-3	Fine Beach Sand	94	1123	12.0	234	2590	11.1	234	2547	10.9	300	4650	15.5
13	1-4	Fine Beach Sand	93	663	7.1	134	2230	16.6	232	2171	9.3	230	4030	17.5
14	1-5	Fine Beach Sand	89	756	8.5	126	2380	18.9	179	1270	7.1	210	2405	11.4
15	1-4 (25-lb. lime)	Fine Beach Sand	124	1064	8.6	140	2260	16.1	215	2571	12.0	218	3760	17.2
16	1-7	Fine Beach Sand	40	362	9.5	61	1460	24	101	709	7.0	139	2100	15.1
17	1-9	Fine Beach Sand	26	99	3.8	...	1420	....	92	811	8.8	131	2150	16.5

Note:—Blanks indicate that there were no specimens left for 6 month breaks.

mortar or concrete which is to be used in construction work?

Nearly all cement is used in compression and compressive strength is the one most important quality that it is called upon to possess. What information does one gather as to the compressive strength of a cement or of the mortar or concrete which may be made from it by making a chemical analysis or by determining its specific gravity or fineness or soundness or setting time? Even if its tensile strength is carefully determined, what do we then know of its compressive strength?

It is often stated that there is a very constant ratio between the compressive strength and the tensile strength of a cement or mortar and therefore if the tensile strength is determined the compressive strength may be computed with reasonable accuracy. This constant ratio is said to range from eight to ten, with an average of about nine. This would indicate that a 1:3 mortar having a tensile strength of 300 lb. per sq.in. would require about 2700 lb. per sq.in. to crush it. There is no doubt that this ratio is very often approximately correct and many tests have been made which corroborate this statement, but that cement mortar and concrete can be made which has twice this ratio between its two kinds of strength is also true and capable of demonstration. In some cases the compressive strength may be as low as four times the tensile strength, while in others the ratio may reach 18 or even more.

As an illustration of the variable nature of this ratio, there is presented herewith a table showing the average results of a series of tests recently made in the laboratory of Westinghouse Church Kerr & Co. with one brand of cement and several grades of sand. There were many different mixtures used, ranging from one cement and three sand to one cement and nine sand. In some cases lime in the form of hydrated oxide was added to the mixture in the proportion of 15 or 25 lb. of hydrate per 100 lb. of cement. The cement used in all the tests was a well known brand imported from Germany.

The sand used was of several qualities. Cow Bay sand screened through a ⅛-in. square-mesh screen was used in some of the mixtures. The same sand with some of the coarser particles removed by screening through a ⅛-in. square-mesh screen was also used. Some tests were also made with standard Ottawa sand. Another series was made with a very fine beach sand obtained from the north shore of Long Island near Whitestone Landing.

The specimens tested were in the form of standard briquettes having a cross-section of 1 sq.in. at the breaking point for the tensile test and 2-in. cubes for the compression test. Two methods were used in making the mortar. In one series the mortar was mixed by hand and applied by hand with a trowel. In the other series the mortar was applied by means of a machine. In both cases the product represented a quality such as should be obtained under fair conditions in the field under the two methods.

The accompanying table shows in Column 1 the number of the mix as recorded in the records of the test. In Column 2 is shown the proportion of cement to sand and in the cases of mixes Nos. 2, 3, 5, 6 and 15 there is shown the amount of hydrated lime added per 100 lb. of cement. In Column 3 is given the kind of sand used in each mixture. Columns 4 to 9 inclusive relate to the hand-made mortar; the first three of them relating to the 28-day test and the last three to the 6-months' tests. Column 4 gives the tensile strength in pounds per square inch of standard shaped briquettes. Column 5 shows the compressive strength in pounds per square inch of the same kind of mortar in 2-in. cubes. Column 6 gives the ratio to the nearest tenth of a unit, that the compressive strength bears to the tensile strength at the age of 28 days. It may be noted right here that the ratios in this column vary from 4.4 to 12. Columns 7, 8 and 9 give the same data for the 6-months' tension and compression tests that Columns 4, 5 and 6 give for the 28-day test. The ratios in Column 9 vary from 10.1 to 24. Columns 10 to 15 inclusive show the results obtained with machine-made mortar, the first three columns relating to the 28-day test and the last three to the 6-months' tests. The ratios in Column 12 average about the same as, or very little higher than, those in Column 6. The average of Column 6 is 7.8, while that of Column 12 is 8.5. Column 15 gives ratios which compare on the average with those given in Column 9, but are somewhat higher as in the case of Column 12 compared with Column 6. The average of the ratios in Column 15 is 15, while that of Column 9 is 13.3, omitting the three tests of standard sand, which do not enter into the average of Column 15 for the reason that no 6-months' tension tests were made of mixes containing standard sand. The variation in the ratios of Columns 12 and 15 is almost as striking as in Columns 6 and 9.

The feature it is desired to emphasize by this table is that the relation of compressive strength in tensile is not a sufficiently constant quantity to make it reliable as a guide in judging of the compressive strength of a mixture of cement and sand when only the tensile strength is determined.

In the table given, the lowest ratio of compressive to tensile strength is 4.4 in the case of the 28-day tests of hand-made mortar of mix No. 8. From this low figure they range up to 24 in case of the 6-months' tests of mix No. 16, and they would have gone higher in the case of the 6-months' tests of mix No. 17, were it not for the fact that the briquettes of this mix fell to pieces before being tested.

It is not presumed that this series of tests shows any greater or any less variation than would be obtained in a regular practice of a contractor or general user of cement. If this is so, how can the practice of testing in tension only be continued by those who desire to know the essential qualities of the materials which enter into their structures?

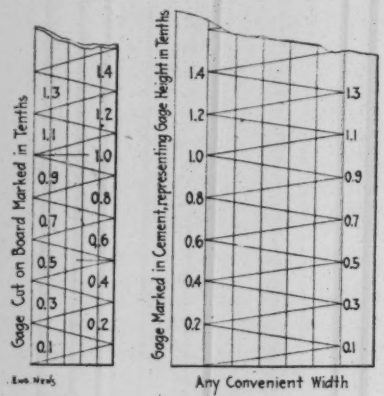
It is time that users of cement mixtures realized the importance of testing their products for the qualities which they must possess in order to properly fulfill their functions in the completed structures.

Government Engineering Work in South Africa, recently authorized, includes totals of some $31,060,000, itemized as follows: Old railways, $10,000,000; new railways, $10,800,000; railway rolling stock, $2,250,000; harbors, $1,825,000; buildings, $3,143,000; telegraphs, $1,850,000; irrigation, $1,192,000.

Measuring and Recording Devices for Irrigation Systems*

By W. G. Steward†

Current Meters—The Gurley current meter No. 621 is generally used by the Reclamation Service and the Geological



F16.1 F16.2

Figs. 1 and 2. Steward Water Gages, Boise Project

Survey for the measurement of canals and rivers. This meter is arranged to indicate through a telephone receiver every fifth revolution of the wheel. An extra recording attachment can be obtained by which each revolution will be recorded if desired. This meter is adapted to all kinds of stream measurement, from the largest rivers to small laterals, wherever there is enough current to turn the wheel.

Canal Gages—The necessity of accurate gage readings in connection with canal measurements led to the designing of the type of gage now generally used on the Boise project. It is one which enables the gage reader to read directly to fiftieths of a foot. The design is adaptable to a vertical or an inclined position. (See Figs. 1 and 2.) It can be cut in wood or marked in the soft cement on the side of a rating flume or cement-lined canal. Several of these gages have been marked in cement on various canals in the Valley and are giving entire satisfaction. They are durable, easily read, and accurate. The gage can be any desired width, it being only required that the vertical lines be parallel and equidistant from each other and that the inclined cross lines be equally spaced. The marking on a board gage consists of saw cuts or channels about ⅛ in. wide and ⅛ in. deep. The figures are cut with a carving tool. Markings thus made will last as long as the board lasts. When the gage is made in cement the lines and figures are cut or stamped into the cement when it is partially set.

Weir Gages—Two new types of weir gages have been designed for use on the Tieton Unit of the Yakima project, copies of the drawings of which are shown by Figs. 3 and 4. These gages will probably give good satisfaction.

Automatic Gages—There are many makes of automatic gages which are more or less in use in the United States. The following are some of them:

Frieze gage, manufactured in Baltimore.

Leitz gage, manufactured in San Francisco.

Lallie gage, manufactured in Denver.

Barrett & Lawrence gage, manufactured in Philadelphia.

Gurley gage, manufactured in Troy, N. Y.

The first four of these consist of a combination of clock- and float-driven mechanism and the record consists of a line drawn across a sheet of paper which is laid off in days and hours in one direction and gage height in the other. The records are taken for from seven days to one month continuous duration.

The Gurley gage is a combination of a clock-driven mechanism, an electric winding device, a set of type wheels moved by the clock and float and a weight-operated printing arm or hammer which prints the indicated time and gage height on an intermittent moving strip of paper at certain periods of time. This gage will run for three months or a year without attention, if properly installed.

An inexpensive and effective auto-

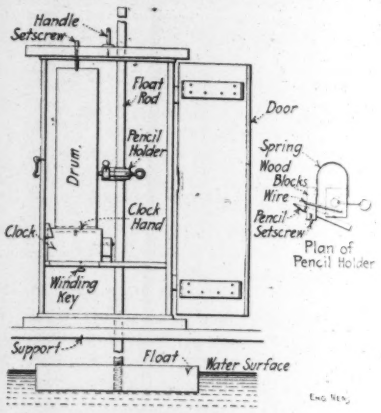
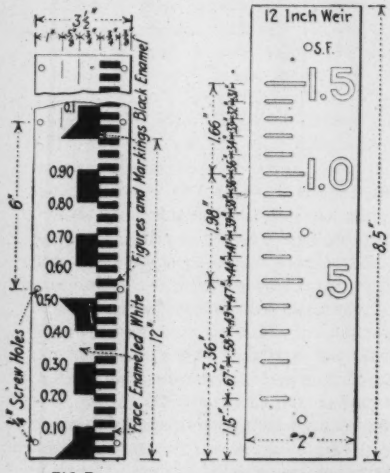


Fig. 5. Automatic Gage Used on Boise Project, Idaho; F. E. Weymouth, Supervising Engineer; F. A. Hanna, Project Engineer



F16.3 F16.4

Figs. 3 and 4. Weir Gages, Designed for Tieton Unit, Yakima Project; C. H. Swigart, Supervising Engineer

(Specifications for Fig. 3: Gages are to be made of No. 18 sheet steel, 3½ in wide, with two lines of ¼-in. holes spaced approximately 6 in. c. to c., and are to be covered with six coats of waterproof enamel of the pattern shown. Graduations are to be "sharp" and accurate, each to be 0.02 ft. The finish and durability of these gages shall be guaranteed to be equal to the best commercial quality of similar enamel work used for street signs and display purposes.

Specifications for Fig. 4: Gage markings must be sharp and accurately made to dimensions and spacing shown on the drawings. Letters and numerals should be of the approximate style and dimensions shown. Material used may be enameled iron with black markings on a white background, or may be cast aluminum with markings depressed about $\frac{1}{16}$ in. and painted black, and face of gage polished.)

matic gage can be constructed by any one having a few tools and a little mechanical ability. This gage, Fig. 5, consists of a box provided with a door and a shelf. On the shelf over a hole is mounted a small cylindrical clock placed on its back. The minute hand of the clock is removed and the point of the hour hand is turned at right angles to the face of the clock. Over this clock, with its center over the end of the clock shaft, is mounted the cylinder or drum and the bent point of the hour hand is inserted into a small hole in the end of the drum. The upper end of the drum is held in place by a setscrew. This drum is made of two circular pieces of wood of any desired dimensions on which is mounted a roll of pasteboard or tin. Passing through the box parallel with the axis of the drum is a float rod to which is attached by means of a screw eye a pencil holder. To the lower end of this rod is attached a wood float. This float is held from turning by means of wire pins that may be passed through holes in the float and stuck into the ground. The float rod should be considerably longer than the box so as to allow for depth of stilling box and movement of float.

In the operation, the gage is placed on a board over a well or stilling box at one side of the weir pool. The water in the pool is admitted to the well or box by a small pipe or hole in the side of the box. The pencil holder is placed on the gage rod so it will record the indicated gage height of the water over the crest of the

*A paper read before the Boise Conference of Operating Engineers for Irrigation Canal Systems Located in Idaho, Oregon and Washington, November, 1911.

†Assistant Engineer, U. S. Reclamation Service, Boise, Idaho.

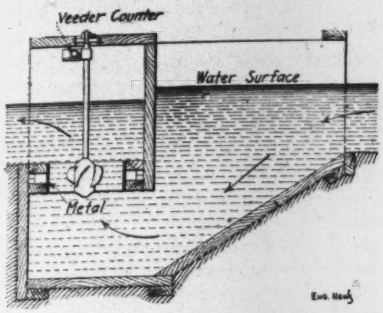


FIG. 6. HILL METER, BOISE PROJECT, IDAHO

weir. The record paper on the drum is marked horizontally in hours and vertically in feet and inches, or in feet and tenths. It is better to have no numbers indicating inches or tenths on the paper till the gage is set and then number the lines as desired, as this will give greater range of gage movement for a given length of cylinder and record paper. This gage can be built for $4 or $5, and with intelligent use gives good results. The drum revolves twice a day and a record sheet can be used two or more days.

AUTOMATIC REGISTERS—At the present, the Service is experimenting on two automatic registering or integrating devices for the measurement of water in canals. The one known as the Hill meter, which is a modification of the Grant and Mitchel patents; the other is the Hanna meter.

The Hill meter, Fig. 6, consists of a box or heading constructed so as to form an inverted siphon, in the discharge end of which is suspended a movable shaft on which is mounted a head with wings or flukes. To the upper end of the shaft is connected a device which registers the revolutions of the head which is turned by the water as it passes through the box. As the rating of this type of meter head is a straight line, the recording device is so arranged that the result is integrated and given in the recorder in acre-feet or some known factor thereof. This meter within the range of its capacity will give reliable results. The cost of its installation should not be great. It is adapted to certain sized laterals having a comparatively steady flow and as there are four sizes of meter made they cover most conditions.

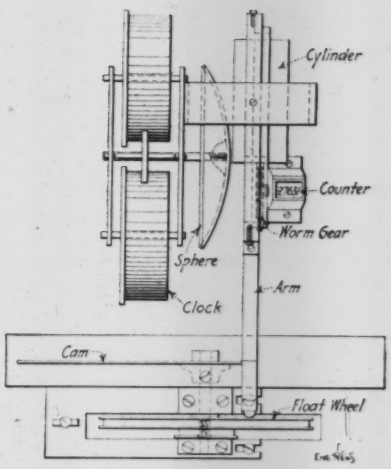


FIG. 7. HANNA METER, BOISE PROJECT, IDAHO

The Hanna meter, Fig. 7, consists of a clock, a spherical planimeter attachment and a float-driven cam. The cam is made to correspond to a discharge curve of a weir, a heading or a stream rating. The clock drives a section of a sphere which in turn drives a cylindrical drum mounted on a shaft. This shaft drives a recording device. The cam is moved by the float and float wheel and in turn moves the arm of the cylinder so as to make the point of contact between the cylinder and the sphere such that the integrated record will correspond to the weir discharge during the period of its operation. The record will be read in acre-feet or some known factor thereof. This meter is applicable to the recording of any discharge where there can be made a discharge curve. It can be used on weirs, submerged or open orifices, rating flumes, or open channels.

SUBMERGED ORIFICE—In ditches where there is not enough fall to make a good weir measurement, the submerged orifice has been used with good results on the Marysville project. Fig. 8 shows the details of the construction of the measuring box and a special slide gage used.

To get the discharge, the runner on the body of the gage is set at zero. The point

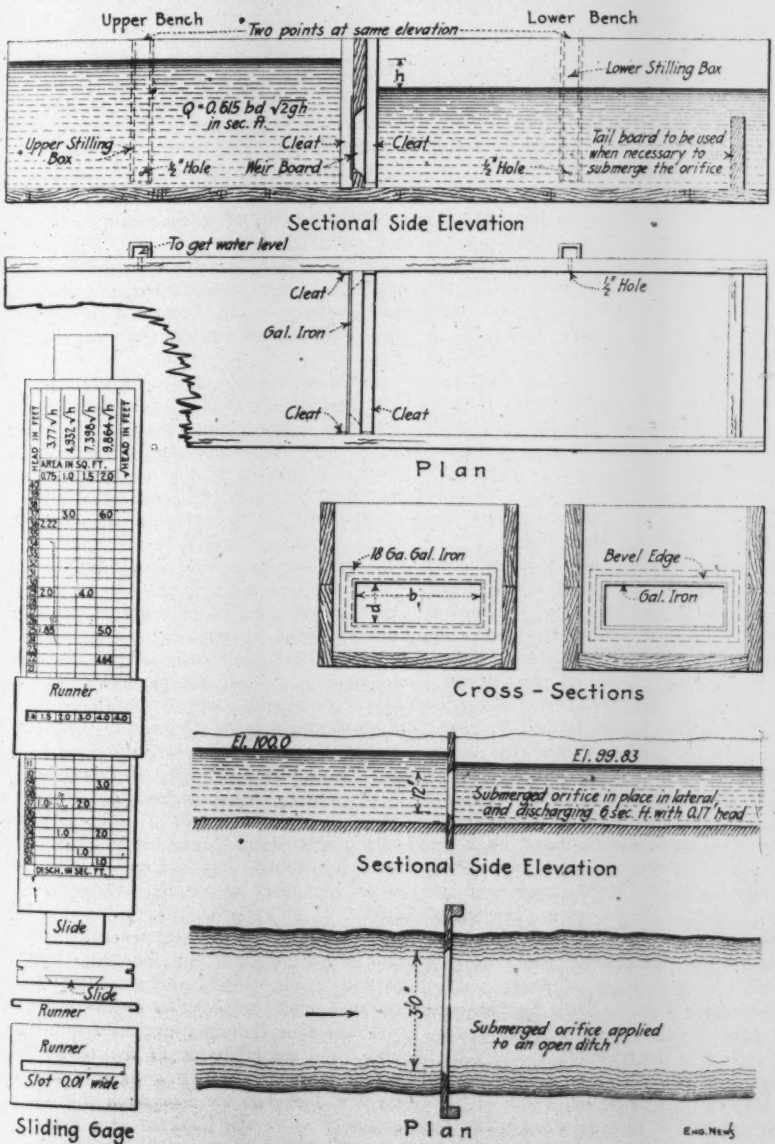


FIG. 8. SUBMERGED ORIFICE AND GAGE FOR OBTAINING HEADS, MARYSVILLE PROJECT, MARYSVILLE, IDAHO

of the gage is placed on the upper bench and the slide is run down until it touches the water surface in the stilling box. The nthe point on the runner is placed on the lower bench and the body of the gage is pushed down until the slide touches the water surface. The water pressure is then read on the left-hand side of the slot in the runner and the discharge is read in column which corresponds to the size of the orifice. For small heads this measurement must be made very accurately.

THE TWIN FALLS MEASURING GATE—This gate is patented by W. A. Buis; A. M. McPherson, Boise, Idaho, is general agent. The device is used on the Twin

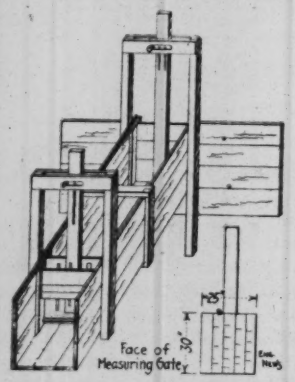


FIG. 9. TWIN FALLS MEASURING GATE, TWIN FALLS, IDAHO

Falls project with considerable success. (See Fig. 9.) It consists of a heading with an upper and lower gate. The upper gate is used to regulate the flow. The lower gate is used to regulate the discharge opening and to indicate the discharge. On this gate are four columns headed 25, 50, 75 and 100, respectively, which numbers indicate the size of the gate opening at which the gate may be set. In the columns below are marks, and on these marks are given the discharge in inches through the gate opening when the water surface stands at the indicated mark. The water below must have a free get-away. Where there is backwater, flashboards must be placed under the lower gate raising the opening until there is free flow of the water. The gate may be made in any width and for any desired height of opening.

Condemnation of Property for Streets—Authority conferred upon a municipality to exercise the power of eminent domain to take private property for the purpose of laying out streets has been held, by a recent Illinois court decision, not to include by implication the authority to condemn land owned by a county and used for a poor farm. Another recent decision in the same state holds that the charter authority to widen streets does not empower a municipal corporation to condemn for such purpose property already devoted to public use, such as for a public library.

A Method of Contract Payments Based on Cost

BY GEORGE I. BATTELLE*

The method of payment for heavy grading, as in railway work, where classification is the basis of settlement, has long been recognized as a makeshift and merely an attempt to base payment upon cost. This method results in more or less friction on almost all contracts, for there are generally some strata in a section of grading which, strictly speaking, may be classified under the specifications as belonging to a formation which calls for a less payment than the actual cost of doing the work, with the result that the contractor justly feels that equity has not been done him in the matter. This condition obtains more generally than its reverse, wherein the work is thrown into a higher classification than that warranted by its cost and the contractor takes an excessive profit.

In some cases recourse is had to force-account as a means to obviate the above difficulties, but this means is usually undertaken on minor work, or if of any size, with a contractor in whom the engineer has confidence. Force-account work, however, lends itself too readily to the undue benefit of the contractor and is not generally resorted to except on small work coming up in connection with a large contract. Force-account work is merely cost plus a certain percentage. Another method of contracting is cost plus a fixed sum. This has not been used in railroad work, but is merely mentioned as an attempt to make the interests of the two parties to the contract as nearly identical as possible when referred to the cost standard.

General usage, therefore, prescribes for railroad work a contract which is based on cost under the guise of geological segregation with numerous tests and conditions to be met as the determining factors in doubtful cases of classification. The interval wherein honest differences may wander is generally of a larger scope than should be allowed, but under the classification idea can not be escaped from.

In what follows will be presented a method that is definitely based upon the actual cost of doing the work with nearly all incentive to pile up the cost removed. Under this method it is to the interest of the contractor to keep the cost as low as possible, for the lower his cost the greater his percentage of profit and at the same time the less is the total amount he is paid, thus reaching ends desired by both parties to the contract.

There are two variations in the use of the method, the first of which is primarily adapted to railway work, while the second is applicable in all branches of construction.

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FIRST APPLICATION

The first method would apply to a railway-grading contract of considerable extent wherein there are various graduations of strata differing in cost of handling. The procedure is as follows:

(1) Estimate as closely as possible the average unit cost of handling the major portion of the material.

(2) Establish an upper and a lower figure of which the average estimated in (1) is the mean with the upper figure twice as great as the lower; i.e., the upper figure is $^4/_3$ the average in (1) and the lower figure $^2/_3$ that average.

(3) For each section of the work the contractor is to get a unit price equal to the actual unit cost of that section plus one-half the difference between that cost and the upper figure established in (2); provided

(4) That the profits be not less than a certain minimum percentage or more than a certain maximum percentage established beforehand.

Should the bulk of the material be estimated to cost less than the remainder, the price schedule would be made an ascending one and a descending one if the reverse was the case. A table can then be easily prepared showing payments of increased percentage profits with decrease of cost below the estimated cost and decreased profits with increased cost.

For illustration, suppose the estimated cost of doing the bulk of the work under a certain grading contract to be 15c. per cu.yd. with the remainder of the work costing more. It is decided to allow a minimum profit of 10% and a maximum profit of 40%. The price schedule under these conditions would be an ascending one of 10c., 20c., 40c., 80c., $1.60, and Table I shows the unit price to be paid, with its attendant profit, when the actual cost has been determined.

Thus if the actual cost is the same as estimated, i.e., 15c., the pay price would be 17½c., showing a profit of 16⅔%; but if the actual cost be cut down to 13c., the contractor would be paid 16½c., but his profit would be 27%. On the other hand, if the actual cost amounted to 17c., the payment of 18.7c. would give him only a 10% profit, which profit would continue until the actual cost ran up to 22½c., when his profit would be 40% on that cost.

Under this method all variations in cost of handling the same geological formation under the different conditions in which it might be found or of uncovering and moving a formation not contemplated under the classification of the contract, would automatically bring its own payment and obviate all discussion with the engineer as to the justness of a higher classification, and the skill used by the contractor in meeting the different conditions would determine the size of his profit.

TABLE I. PRICE SCHEDULE 10c., 20c., 40c., 80c., $1.60. MIN. PROFIT 10% MAX. PROFIT 40%.

Cost c.	Deduction from next highest price*	Pay Price c.	Profit c.	Profit %
		11.000	1.000	10
10	.....	11.550	1.05	10
10½	.....	12.100	1.10	10
11	.....	15.750	4.250	36.96
11½	4.250	16.000	4.000	33.33
12	4.000	16.250	3.750	30.00
12½	3.750	16.500	3.500	26.92
13	3.500	16.750	3.250	24.07
13½	3.250	17.000	3.000	21.44
14	3.000	17.250	2.750	19.00
14½	2.750	17.500	2.500	16.67
15	2.500	17.750	2.250	14.52
15½	2.250	18.000	2.000	12.50
16	2.000	18.250	1.750	10.60
16½	1.750	18.700	1.700	10
17	1.300	19.250	1.750	10
17½	0.750	19.800	1.800	10
18	0.200	20.350	1.850	10
18½	.....	20.900	1.900	10
19	.....	21.450	1.950	10
19½	.....	22.000	2.00	10
20	.....	23.10	2.10	10
21	.....	24.20	2.20	10
22	.....	31.50	8.50	36.96
23	8.50	32.00	8.00	33.33
24	8.00	32.50	7.50	30.00
25	7.50	33.00	7.00	26.92
26	7.00	33.50	6.50	24.07
27	6.50	34.00	6.00	21.44
28	6.00	34.50	5.50	19.00
29	5.50	35.00	5.00	16.67
30	5.00	35.50	4.50	14.52
31	4.50	36.00	4.00	12.50
32	4.00	36.50	3.50	10.60
33	3.50	37.40	3.40	10
34	2.60	38.50	3.50	10
35	1.50	39.60	3.60	10
36	0.40	40.70	3.70	10
37	....	41.80	3.80	10
38	....	42.90	3.90	10
39	....	44.00	4.00	10
40	....	46.20	4.20	10
42	....	44.40	4.40	10
44	....	63.00	17.00	36.96
46	17	64.00	16.00	33.33
48	16	65.00	15.00	30.00
50	15	66.00	14.00	26.92
52	14	67.00	13.00	24.07
54	13	68.00	12.00	21.44
56	12	69.00	11.00	19.00
58	11	70.00	10.00	16.67
60	10	71.00	9.00	14.52
62	9	72.00	8.00	12.50
64	8	73.00	7.00	10.60
66	7	74.80	6.80	10
68	5.2	77.00	7.00	10
70	3.0	79.20	7.20	10
72	0.80	81.40	7.40	10
74	....	83.60	7.60	10
76	....	85.80	7.80	10
78	....			

*Equals one-half difference between cost and next highest price.

TABLE II.

Suppose the price bid as the actual cost to be 12c. and the min. and max. profits 5% and 30%; 16c. would then be the figure to which the following table is built to fulfill the conditions.

Cost	8	8½	9	9½	10	10½	11	11½	12	12½	13	
Pay	10.4	11.05	11.70	12.35	13	13.25	13.50	13.75	14	14.25	14.5	
Profit	30%	30%	30%	30%	30%	26%	23%	19.6%	16.67%	14%	10.46	
Cost	13½	14	14½	15	15½	16						
Pay	14.75	15	15.225	15.75	16.275	16.800						
Profit	9.26%	7%	5%	5%	5%	5%						

The jumps in the table between 11 and 11½c., 22 and 23c., and 44 and 46c. correspond to, though they are not so great as, the variations of price under the usual classification contracts. It is near these points that the only incentive to increase cost exists and where the engineer in charge of the work would have to carefully watch the cost account of the contractor and be sure that he was neither lying down on him or trying to slip in outside costs in order to slightly raise his cost to that allowing the higher percentage. With a carefully estimated cost to which the price schedule is adjusted this contingency is quite remote.

In the measurement of volume to determine cost, individual cuts could be taken by themselves should the same method be taken of working them throughout, instead of measuring the volume of the different strata. Total yardage of a cut into total cost of removing it would point out its own payment and be an easier solution of the price to be paid than long bickering between engineer and contractor with a probable lawsuit showing on the horizon over difference in judgment as to the proper classification to be allowed for some certain yardage.

The variation of this cost method of payment referring to a single cost schedule where each figure therein is double its predecessor is applicable to any piece of large work composed of varying units, since in the railway grading for which it is written the cost of doing a unit portion widely varies by reason of uncontrollable conditions and may be anything between two extreme minimum and maximum prices.

Second Application

The price schedule under this variation is determined as under the first and is applicable to all forms of construction, as well as railway work.

Using railway work for an example, instead of asking for a bid for which the contractor is willing to remove certain kinds of material, a bid is sought upon the estimated cost of doing the work by different methods, i.e., by scrapers, by wheelers, by steam shovel and wagon, by steam shovel and cars, or by whatever method the contractor is equipped for and the situation is favorable to. The difference between the two applications lies in the fact that in the first the bid is based upon a rather hazy estimate of the different priced yardages to be handled which is often very misleading. In the second case the method of handling a closely known yardage is the basis of the bid, with each party sharing in the decreased cost if the estimated cost is found to be too large.

Corresponding to the figure bid for each of the different methods proposed for handling, a table of payments could be made by referring the actual cost of any certain yardage to a figure one-third larger than the bid and dividing this difference between both parties to the contract up to the points where the minimum and maximum percentage were reached, beyond which points their respective percentages would obtain on the actual cost of doing the work. This may be illustrated in Table II.

The second variation is merely a special case of the first. In the first, one price schedule is used throughout for all methods and final costs and is obtained by doubling each preceding figure to obtain the succeeding one; in the second, a single short table is made for each figure bid and though the cost may increase it never jumps into the next higher table, for the one table always applies to the method of handling upon which it was based.

Though the illustrations of both variations of the method are made by reference to railway grading, the second way lends itself very readily to concrete, steel, or brick work, timber framing, roadwork, pile-driving and in fact to all construction where payment is made upon the number of units of various kinds incorporated in the work.

Under this method the one letting the contract would have to be supplied with, as well as check, all pay-rolls, original bills of material, equipment rental, and stock rental, if not more, to the cost record on a contract as he now gives attention to the workmanship under it.

Among the advantages to be cited for this method of contracting may be mentioned:

The range of bidding on any one contract is larger than under the present, for in addition to the range of prices there is the variations in the minimum and maximum profits to be considered, for while two or more bidders might submit the same figure as to actual cost there might be some considerable difference as to the range between minimum and maximum profits that they would consider equitable.

As the contractor is guaranteed against loss each contract would give the contractor the profits on that contract and he would not be looking to make a succeeding contract stand the loss of a former. This is as it should be, for no matter what the conditions it is not fair to any of the parties concerned to carry a loss from one contract to another. Each contract would be required to meet the labor, material and weather conditions which exist during the completion of that contract. Unforeseen emergencies are provided for at their actual cost instead of being figured as probable and the attempt made to provide therefore by tacking a lump sum upon the figure for which, under normal conditions, the contractor would be willing to undertake the work. It is not claimed for this method, however, that it leaves nothing to be desired, for under it the incentive would still exist to slur the workmanship in order to reduce the cost.

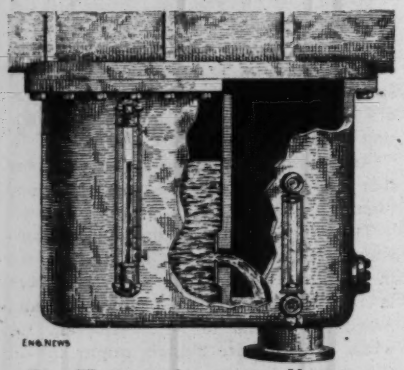
The contract being let at an estimated cost which carries with it a payment allowing a fair profit and the one letting the contract sharing in any saving or

decrease in cost, he cannot in justness feel that the contractor has the best of it if the contractor makes a larger percentage profit, inasmuch as the latter would receive a total sum less than the estimated total. The method is not at all complicated, but is essentially a sliding-scale payment which readily fits into present practice of cost keeping and competitive bidding.

An Indicating Hotwell for Measuring Engine Exhaust

The most favored test method of measuring steam consumption of an engine or turbine is by weighing the condensed exhaust, but in few plants is this measurement carried along as a part of the daily routine since the apparatus employed has been generally too clumsy and the work too inconvenient.

To remove such troubles and to keep the advantages of continually measuring



THE WHEELER INDICATING HOTWELL

the condensate, an indicating hotwell has been designed by the Wheeler Condenser & Engineering Co., of Carteret, N. J., for attachment to the bottom of a surface condenser between the drain and hotwell pumps.

The arrangement is shown in the accompanying figure. The hotwell casting is divided into two compartments which are connected by an orifice in the central wall. The left-hand compartment drains the condenser casing, while the right-hand chamber is drained to the hotwell pump. With an accurately made orifice plate, the coefficient of discharge remains closely constant for different heads and the quantity of condensate discharged is found with considerable accuracy from readings of head on the orifice. The head is read on a gage glass shown, and this is equipped with ball checks to prevent losing the vacuum in case of breaking the gage glass. A second gage is also provided to show the height of water in the hotwell suction compartment where the water must not submerge the orifice for accurate measurement. The first gage may be calibrated to read directly pounds of steam per hour.

It is necessary to calibrate each orifice and gage separately. Under these conditions the accuracy over the whole range is within 2% and over a narrower range, of 75% to 125% normal load, the accuracy is within 1%.

The same result is aimed at in devices shown by other designers, but working at the other end of the power-generating system. In these, the feed water is measured automatically and recorded as it flows, over a weir, from one tank to another. This device may be separate or built in connection with a feed-water heater.

Some Observations on the Disintegration of Cinder Concrete*

BY GEORGE BORROWMAN†

Cinders, as an aggregate in concrete, are used rather commonly where a light, cheap mixture is desirable; also in fireproof construction, and perhaps in the latter capacity this kind of concrete finds its most important application.

Its efficiency, however, has been the subject of considerable discussion. Various tests have been made, the results of which tend to show the superiority of this material over all other fire resisting substances. There are also several patented floors involving cinder concrete which have been used successfully. Nevertheless, it has failed in some instances when employed for ordinary purposes, cracking and disintegration taking place. These discordant results are due, no doubt, as Freitag suggests in his "Fireproofing of Steel Buildings," to the use of cinders of varying quality, but to the writer's knowledge the matter has not been further investigated. A particular case of disintegration, accompanied by a powdery efflorescence on the surface of the concrete, was observed by Dr. S. Avery, now Chancellor, but formerly head of the department of chemistry in this University, who prompted some study of the phenomena and also made various helpful suggestions during the experiments.

Blocks of concrete containing cement, sand and ¼-in. cinders were first prepared, the proportions by weight being 1:2:3, respectively. The cinders were from steam coal and slack and were carefully screened. The cement and sand were of good quality, the latter being sieved through 20-mesh onto 30-mesh screens.

Another lot was then made up using cinders from ½ in. to 1 in. in size, the proportions being 1:1:1. In each case just enough water was used to insure thorough incorporation without running. All the blocks were placed on a glass plate to prevent loss of moisture and covered with a damp cloth for two weeks.

At the end of this period, the specimens having the proportions 1:2:3 showed no change, but were somewhat porous and friable, owing to the relatively large bulk of cinders. A little later, however, fine cracks developed, with general disintegration. The others (1:1:1), though compact and hard, also showed signs of cracking and in a short time the breaks became very pronounced. In each case they radiated from certain points.

These latter specimens were broken open to facilitate examination and in some cases the breaks originated from cinders that showed oxidation of much iron. The surfaces of the concrete bounding the cracks crumbled easily, indicating more or less general weakening.

The appearance of these oxidized cinders suggested the presence, originally, of much ferrous iron, probably as sulphide. The analysis of the cinder stock showed considerable sulphur as sulphide and sulphate; also iron, though the proportion of ferrous iron could not be determined in material of this sort, with accuracy:

ANALYSIS OF CINDERS

		Per cent.
	SiO_2	37.86
Total Fe as	Fe_2O_3	21.16
	Al_2O_3	17.40
	CaO	10.96
	MgO	0.83
	S	0.60
	SO_3	5.82
	Undetermined (C, etc.)	5.37
		100.00

It seemed probable, then, from the foregoing data, that oxidation of iron and sulphur produced internal stress and consequent cracking, aided, no doubt, by the general weakness produced by the diffusion of soluble sulphates. This theory was further strengthened by finding that the efflorescence, found on the surface of the disintegrating concrete before mentioned, was ferrous sulphate.

On this assumption attempts were made to bring about similar results by substituting for cinders artificial iron sulphide, prepared in several ways. These experiments, however, were not successful. On breaking open the blocks, the sulphide was found to be unchanged, surrounded by hard, compact cement. Soluble sulphates being absent in the mixture, the blocks had, of course, greater initial strength than those containing cinders and would have withstood considerable stress had there been any. The lack of oxidation, though, in these tests, indicated that in the case of the cinders the necessary oxygen must have come from the cinders themselves, since they were equally well protected from the air.

This conclusion drew attention to the fact that about one-third of the cinder stock consisted of a very porous coke. As is well known, this substance will

*Abstract of paper in "The Journal of Industrial and Engineering Chemistry," June, 1912.

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persistently occlude large amounts of oxygen. According to Starr and Levis, air-dried coke contains absorbed gases in the proportion of about 0.7 c.c. per gram of coke, 2 to 6% of the gases being oxygen. The oxidation in the concrete was most pronounced in the coke. Moreover, some pieces of the latter were picked from the cinder stock and tested for sulphide sulphur, about 1% being found. One nodule from which cracks radiate proved to be a piece of coke so thoroughly oxidized as to look like a mass of ferric oxide. This was tested and found to contain but a trace of sulphur.

As a final experiment, some of the original stock of cinders were kept exposed to air and moisture until, on washing, they showed the presence of no soluble sulphate. They were then made into a block of concrete similar to those prepared before (1:1:1). Under parallel conditions no disturbance took place, the material retaining perfect stability.

From these tests we may conclude that cinders with much sulphide and sulphate sulphur are likely to give unsatisfactory results, especially if there is much coke or porous material present; also that such material may be improved if allowed to weather, with occasional washing, until the ferrous iron and sulphur have been oxidized and leached out.

Failure of Supports to a Roof Water Tank

By Edward Godfrey*

On Aug. 7, 1912, a wooden water tank on top of the factory building of the Union American Cigar Co., at 28th St., Pittsburgh, Penn., fell through the roof of the building, killing several persons and injuring others.

The writer was called in to make an examination and report on the cause of the wreck. The company has shown a commendable desire to let the facts be known and is making every effort to meet just claims. There are several important lessons for designers in this failure, and hence a publication of the facts as they have already been given to the Coroner ought to be of interest to engineers.

The accompanying sketch shows the conditions before the wreck as the writer has reconstructed them from the ruins by means of marks on the several pieces.

The tank was 10 ft. in diameter and 12 ft. high, having a total weight, when filled, of about 70,000 lb. It had just been completed and had received, on the day of the failure, its first filling (or partial filling) of water. It is said to have had 7 ft. of water in it a few hours before the failure and at the time of the failure less than this, as it was being drawn from.

An old tank was removed and the new one placed on the same steel beams, which were not moved from their original location. No plans were available to show where the new tank had rested, but from measurements on the beams and the timbers supporting the tank and from marks thereon it was found that the most probable position was as shown in the sketch. In fact the whole record of the failure is plainly written in the wreckage.

There is no doubt whatever that the failure was due to insufficient steel support under the tank. There are three phases to the weakness of this steel work.

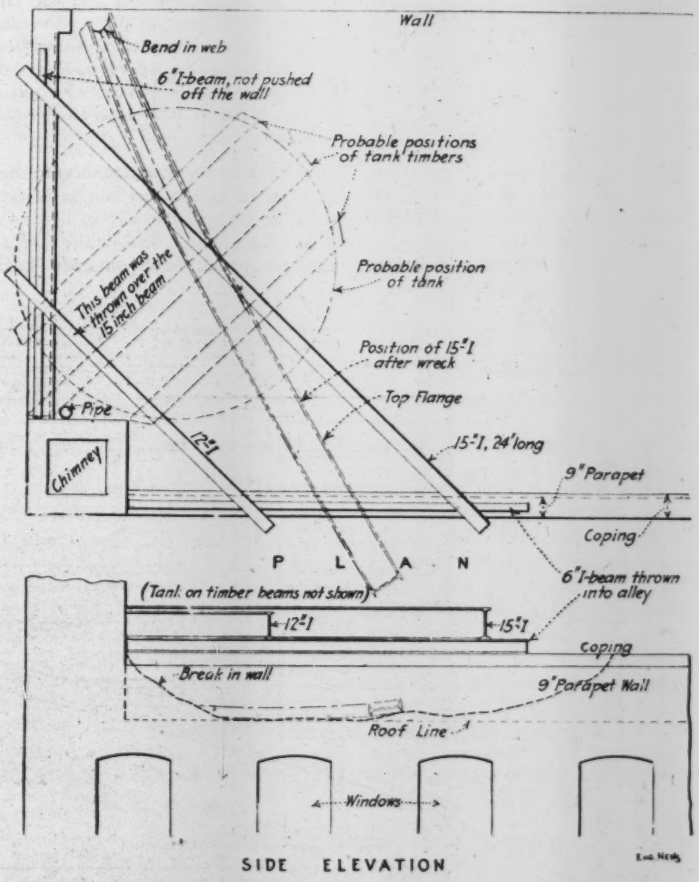
First, the 15-in. I-beam was only about half strong enough for the load it would have been compelled to carry if the tank had been filled, assuming that the top flange was held against buckling, and assuming that the beam was not wasted away by rust. A little calculation will discover this point.

Second, the two beams under the tank as well as the two 6-in. beams on the walls had nothing to prevent them from leaning or flopping over at the ends.

Third, the beams were badly rusted. This is particularly true of the 15-in beam, and the greatest deterioration from rust was at the very point where the greatest strength was needed, namely, in the top flange under the tank. The old tank had rested on some kind of continuous support, and under this the beam had rusted very badly.

Besides this there were other faults in the design. The parapet walls should have been thicker than 9 in. The tank should not have been so placed that the greater part of its weight rested on the 15-in. beam. The tank beams should not have been balanced on single 6-in. beams on the walls; double beams with separators should have been employed.

The sketch does not show fully the shape of the 15-in. I-beam after the failure. There was a heavy sag in it as it lay, the flanges being badly bent and buckled in the portion under the tank timbers. The large twist in the beam and the evidence of its having keeled over at the right-hand end show the reason and nature of the failure. In this failure is a very practical example of the weakness of beams lacking end stiffness and bracing.



Elevation and Plan of Roof Water-tank Support

*Monongahela Bank Building, Pittsburgh, Penn.

Car Shops for an Electric Railway

The design and equipment of repair shops for electric railway systems has made great advances within recent years, and for large plants the requirements are very similar to those for the car shops of steam railways. This is especially the case in view of the great extension of interurban lines and the gradual introduction of cars of steel and combination (steel and wood) construction. The electric railway has opened a new field for the engineer, and this article describes a large shop plant of modern design for an electric railway operating both urban and interurban lines.

With the development of the electric railway in street and interurban service, and with the development in size and power equipment of the electric cars, there has been a corresponding development in the shop facilities for the repair and maintenance of the cars. The cars of electric railways are subject to very severe usage; much of the machinery and electrical apparatus is carried beneath the frames and is therefore liable to be splashed with water and mud, while the upper part is liable to sustain injury from more or less important collisions with other vehicles. There is consequent necessity for adequate facilities for periodical overhauling and repair of the cars, and the introduction of steel cars has necessitated the provision of special machinery and facilities. As a result of these conditions many of the modern shops of electric railways are of large extent, and are built and equipped with a view to first-class work and economical operation.

A good example of plants of this kind is afforded by the new shops of the Milwaukee Electric Railway & Light Co., of Milwaukee, Wis. Information regarding this plant has been obtained during a visit to the plant, while a part of the information given below has been taken from a paper read before the Wisconsin Electrical Association by R. H. Pinkley, Engineer of Way and Structures. It may be noted that the plant covers in all about 10.3 acres, and employs at present about 500 men. It is designed with sufficient capacity to take care of an equipment of 1200 cars (including 240 interurban cars), but at present only about 600 cars (120 interurbans) are provided for. Until the growth of the system requires the plant to work to full capacity a part of it is utilized for car storage. However, new car barns or storage sheds are to be built in the same neighborhood (but on a separate site), so that most of the old storage facilities in the busy part of the city can be abandoned. The general and department offices of the company are in a large modern building of its own in the center of the city, part of the first floor of this being utilized as a passenger station.

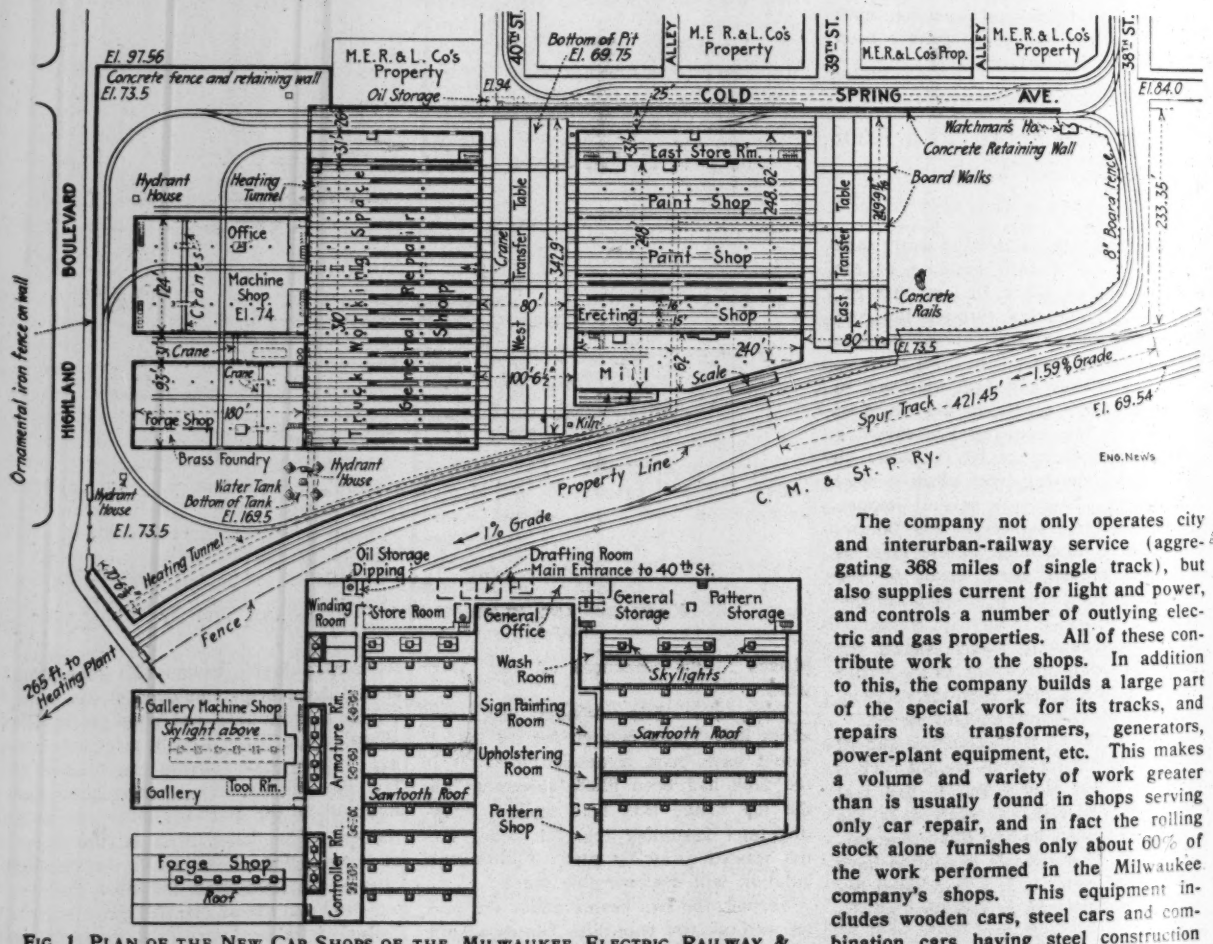


FIG. 1. PLAN OF THE NEW CAR SHOPS OF THE MILWAUKEE ELECTRIC RAILWAY & LIGHT CO.; MILWAUKEE, WIS.

The company not only operates city and interurban-railway service (aggregating 368 miles of single track), but also supplies current for light and power, and controls a number of outlying electric and gas properties. All of these contribute work to the shops. In addition to this, the company builds a large part of the special work for its tracks, and repairs its transformers, generators, power-plant equipment, etc. This makes a volume and variety of work greater than is usually found in shops serving only car repair, and in fact the rolling stock alone furnishes only about 60% of the work performed in the Milwaukee company's shops. This equipment includes wooden cars, steel cars and combination cars having steel construction up to the level of the window sills.

General Description

When it was definitely decided to construct new shops, a study of representative street-railway shops throughout the country was made for the purpose of deciding on the best features to be adopted.

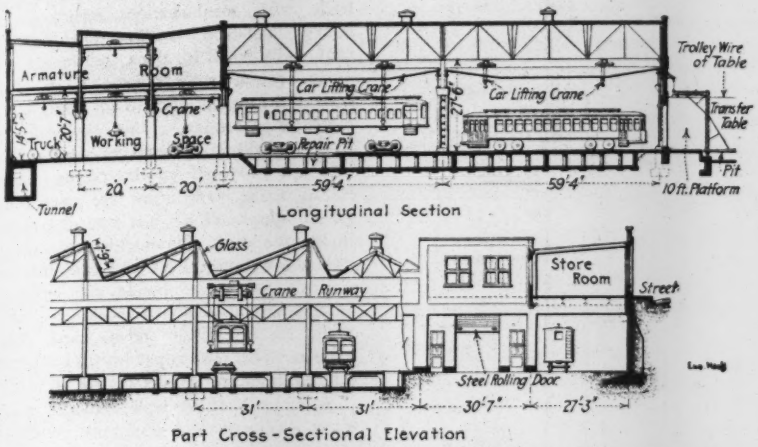


Fig. 2. Longitudinal and Cross-sections of Car Repair Shop, Showing Cranes for Handling Car Bodies

In a majority of cases it was found that these shops had grown up from a small beginning and had been added to as occasion demanded and as seemed best suited to immediate requirements. In the more modern plants, a wide difference was found in general arrangement and in all details, depending on local conditions. Some (where land was not very valuable) had isolated buildings scattered over a large territory; others where space was limited were built compactly. The same difference extended through all of the details of construction and arrangement. Most of these plants have excellent individual features, but each has been developed to suit its own peculiar requirements and limitations.

The new shops (known as the Cold Spring shops) are built on an entirely new site, acquired for the purpose, so that their design was not hampered by considerations of existing facilities on the site. They are conveniently located with relation to the street-railway system, so that cars from the various lines can reach them easily, but they are in an industrial district beyond the congested part of the city. The general considerations which decided the selection of this location of the shops were as follows: 1, central location, saving mileage on dead cars; 2, accessibility to steam-railway connections; 3, low cost of land, on account of its being depressed and adjacent to railway. The shops were completed and put in service in December, 1911.

Fig. 1 is a plan of the general layout of the plant. The site originally was occupied by a brick yard and was graded down to an almost uniform level over the entire area. The upper yard has an area of about 10.3 acres, and is about 10 ft. below street grade at 38th St. and Cold Spring Ave., and 26½ ft. below the grade of Highland Boulevard. At the southeast corner the property is practically level with the C., M. & St. P. Ry. tracks, while from this point to the southwest corner, there is a drop of about 23 ft. in the grade of the ground adjacent to the railway tracks. This necessitated the erection of a retaining wall along the west and north sides, and another along the south side. The former slopes from a height of 7 ft. at 38th St. to a maximum of 26½ ft.; the latter begins at 6½ ft. and increases to 26½ ft. at the southwest corner. These walls are of reinforced concrete, and aggregate about 2120 ft. in length. Approximately 4100 cu.yd. of concrete were used. In general all walls over 20 ft. high are of the counterfort type, and those under 20 ft. high are of the cantilever type. A number of forms of cross-section were necessitated by the different conditions encountered. All of the walls were calculated for an equivalent horizontal hydraulic pressure of 33 lb. per foot of height. Tension in the steel was limited to 16,000 lb. per sq.in. and compression in concrete to 650 lb. per sq.in. These walls inclose the property on three sides, leaving a space along the east and south sides which is closed by a tight 8-ft. plank fence, surmounted by barbed wire, and having gates for the entrance tracks.

The main track entrance for the electric cars is at the northeast corner. The right-hand or incoming track is carried down at a 3% grade to the general level, and from it cars can enter on the north ends of the transfer tables or can be taken entirely around the loop track and out on the south track. Steam-railway cars can be brought in over the track at the southeast corner of yard, and placed in any part of the grounds or buildings by means of the transfer tables. The two tracks on the south side lead down under the viaduct to the company's storage yard which is located west of the boulevard. This yard provides for storage of heavy material, and for such maintenance and construction equipment as tower or line-repair cars, snow plows, derrick cars, steam and electric shovels, traveling concrete mixers, dump cars, etc. In this yard, also, is located the heating plant noted below.

The buildings cover an area of about four acres and the total floor area is about five acres. The buildings are arranged so that work passes through them in a direct route. Blacksmith work, brass-foundry work and machine work occupy the west buildings. Truck work and iron work on the car bodies is done in the west end of the repair shop. Armature and controller work is done on the gallery above truck space. Across the transfer table are located the dry kiln, planing mill, body shop and paint shop. The pattern shop and space for

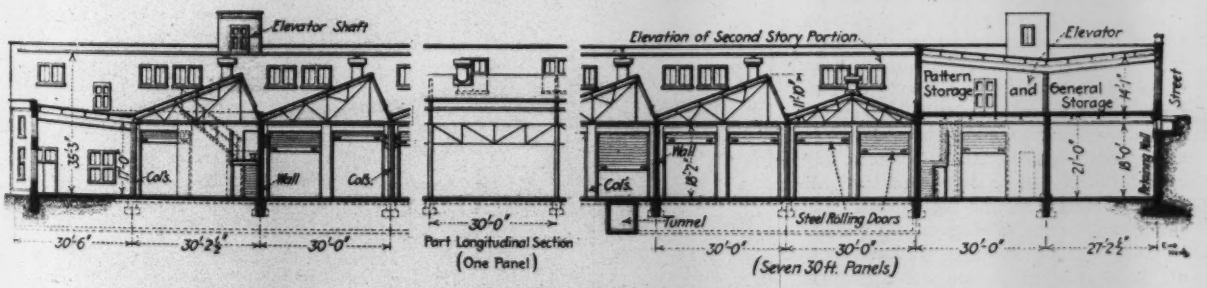
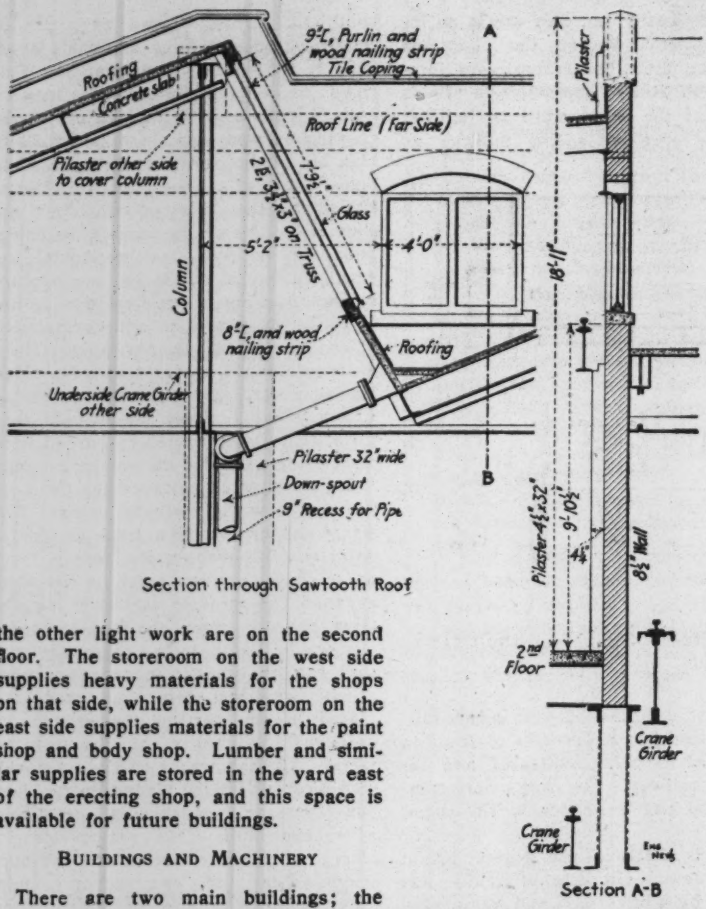


Fig. 3. Part Sections of Paint Shop



Section through Sawtooth Roof

FIG. 4. SAWTOOTH-ROOF CONSTRUCTION, WITH DOWNSPOUTS INSIDE BUILDING

the other light work are on the second floor. The storeroom on the west side supplies heavy materials for the shops on that side, while the storeroom on the east side supplies materials for the paint shop and body shop. Lumber and similar supplies are stored in the yard east of the erecting shop, and this space is available for future buildings.

BUILDINGS AND MACHINERY

There are two main buildings; the general repair shop, 340x180 ft., and the paint and erecting shop 280x240 ft. These are on opposite sides of an 80-ft. transfer table, the buildings being 100 ft. apart. A similar transfer table at the east side serves the present storage yard and will serve the buildings which are to be erected in this yard when required. Beyond the west building are the machine shop, 124x180 ft., and the forge shop, 93x180 ft. The offices are at the street level, with the main entrance opposite 40th St.; on the same level are the upper floors of the shops. The buildings are of steel-frame construction with concrete footings and floors. They have 13-in. brick walls (with a facing of buff brick), concrete window sills, and monolithic concrete roof slabs covered with a waterproof composition of pitch and gravel. Steel rolling doors are used for all large entrances, and the windows and skylights have wire-glass in metal frames. The two main shops have roofs of the sawtooth type, as shown in Figs. 2 and 3. The construction is fireproof throughout. The floors on the ground-floor level are of 1:3:5 concrete, 6 in. thick; this is laid on cinder filling, and has a ¾-in. cement finish (1:1½). The blacksmith shop, however, has a cinder floor and in the west part of the machine shop there is a heavy plank floor on concrete foundation. The galleries and second floors are of reinforced concrete laid on top of the steel beams. These floors are 4½ in. thick (with cement finish) for working spaces, and 4 in. for offices. They are designed in general for a live load of 200 lb. per sq.ft. Stairways are built with steel-channel stringers and heavy cast-iron treads. The concrete roof slabs are 2 in. thick, reinforced with ⅜-in. round rods spaced 6 in. apart across the spans and 18 in. apart in the opposite direction. The skylights are fixed and 30 in. galvanized-steel ventilators are provided on the ridges. About 85 of these ventilators are used. The gutter boxes are of cast iron and project inside the roof. Downspouts are of galvanized wrought iron and cast iron and are all led down inside the buildings. In this way it is hoped to obviate trouble from frozen gutter boxes and downspouts. The roof construction and drainage system are shown in Fig. 4.

A special effort was made to secure good drainage and plumbing conditions. All plumbing fixtures are of the most substantial type, designed to withstand the roughest kind of usage. Toilet and locker rooms are located on galleries wherever possible and are distributed throughout the buildings at convenient points.

There are five electric elevators, each of 4000 lb. capacity, located at convenient points for transferring material between the first and second floors. Electric traveling cranes span each of the three bays of the machine shop and blacksmith shop. Similar cranes with special appliances for lifting car bodies

FIG. 5. CAR-HANDLING CRANES IN REPAIR SHOP; MILWAUKEE ELECTRIC RAILWAY & LIGHT CO.

span the two east 60-ft. bays of the repair shop. Smaller cranes are placed in the three bays over the truck-working space, and in the two bays of the armature department on the second floor. The heavier cranes in the center bays of the forge and machine shops, and the car-

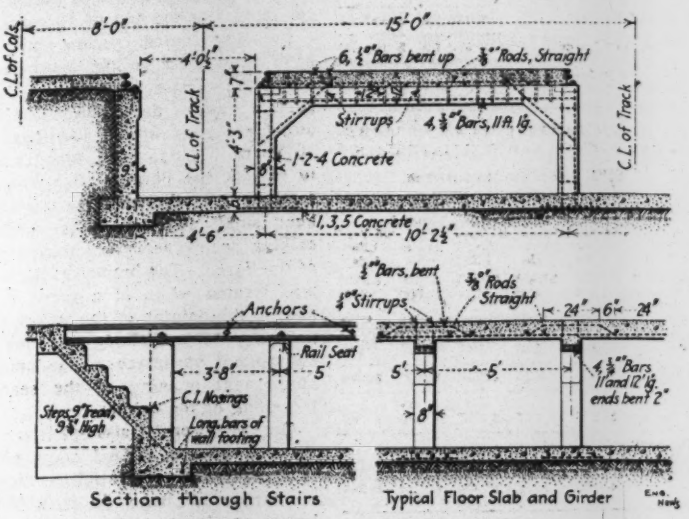


FIG. 6. DETAILS OF PITS IN CAR REPAIR SHOP

body lifting cranes, are operated from cages. All other cranes are operated from the floor by means of pendant cords attached to the motor controllers. With the car bodies raised by the cranes in the front bay, the trucks can be pushed over into working bays under the three crane runways, as shown in Fig. 2. It will be seen that in the car-lifting cranes the special feature is that instead of the usual single trolley, with central hoist, there are two trolleys, each with two hoists carried outside the girders and far enough apart to allow a car to stand between them. The hoists can then be attached by slings to beams placed under the car. Fig. 5 shows one of the cranes carrying a car body which has been lifted from the trucks.

The car pits are of the open type, and built entirely of reinforced concrete. The posts are 8 in. square and are located 5 ft. apart in the direction of the rails. The track centers are alternately 15 ft. and 16 ft., apart. The rails are of 7-in. 95-lb. T-section, and span from post to post. The floors between pits have the same depth as the rails (7 in.), and span from beam to beam in the direction of the rails. The spaces beneath them are utilized for storage and tools. Fig. 6 shows the construction of the floors and pits, and Fig. 7 shows the method of supporting and anchoring the 7-in. girder rails of the pit tracks.

The transfer tables have pits 45 in. deep, but the rails are only 21 in. below the floor level, being carried on concrete longitudinals and on ledges of the side walls. In this way the obstruction of the tracks by snow is prevented, and no trouble of this kind was experienced even during the heavy snowstorms of the past winter. The pit is well drained. Current for operating the table is taken from an overhead trolley wire strung along the wall of the building. A trolley wire is strung over the table, and hinged brackets enable connection to be made with the trolley wires of the shop tracks.

HEATING AND FIRE PROTECTION

A direct, low-pressure, vacuum steam-heating system is used, and the air pumps handle both the water and air through a single return system. The buildings are heated mainly by pipe coils, a part of which are placed along the walls under the windows and the remainder in the lower chords of the roof trusses. The heating plant is located in the yard west of the viaduct, and a concrete-lined tunnel (shown in dotted lines on Fig. 1) connects this plant with the main buildings. The tunnel is about 1875 ft. long. It is 6 ft. wide and 6½ ft. high inside, with roof and walls reinforced according to the depth below the surface and the loads on the surface. In the tunnel are carried all the heating, water and air pipes; also the light and power feed wires.

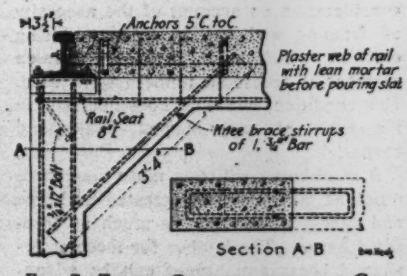


FIG. 7. TRACK CONSTRUCTION ON CAR REPAIR PITS

For fire protection, all parts of the buildings (except the blacksmith shop, machine shop and truck-working space) are provided with a standard overhead dry-pipe sprinkler system. There are also 50 standard 1½-in. hydrants, each fitted with 50 ft. of hose and nozzle; these are spaced about 120 ft. apart throughout the buildings. In the yard there are nine standard two-way fire hydrants; each is in a standard hose house with full equipment, including 150 ft. of 2½-in. hose, standard nozzle, and tools. In addition there is an equipment of fire pails and hand chemical extinguishers. South of the repair shop is a 100,000-gal. gravity tank for the fire-protection system; this is supported on a steel tower and its bottom is 96 ft. above the ground. The tank supplies water through two 8-in. pipes laid in the tunnels, leading to all the buildings and forming a complete loop.

LIGHT AND POWER

The current for lighting and power for the shops (except for elevators, cranes and transfer tables) is supplied from the company's 60-cycle distributing system. A special 4000-volt, three-phase feeder leads from the Commerce St. power plant to the transformer room at the shops, which is centrally located just west of the office. Power for the motors is supplied by two 400 kw., 60-cycle, three-phase, 4000-240 volt transformers, and is distributed from these to the power panels located in the different departments. Current for lighting is taken from one phase of the feeder through two 25-kw. 2300-110-volt single-phase transformers, and is distributed to lighting cut-out boxes in the various buildings. One transformer of each kind is in reserve. All wiring is laid in conduits. Direct current for operating traveling cranes, elevators and transfer tables is taken from the 600-volt trolley system. The motors on traveling cranes aggregate 460 hp.; on the elevator motors, 170 hp., and on the transfer tables 70 hp.

The shop machines are motor driven, the heavy machines having individual motors and the lighter ones having group drive. The connected power of alternating-current motors for the various departments is as follows:

Forge shop	121¼ hp.
Machine shop	439½ hp.
Armature room	18½ hp.
Mill	121 hp.
Pattern shop	32 hp.
Total	732½ hp.

The Mazda tungsten lamps are used for illumination throughout, and lighting in general is from overhead lamps uniformly spaced. The quantity of light for each class of work has been determined very carefully. It was decided that individual machine and bench lighting was undesirable except in a very few instances, and by working up comparative illumination tables for carbon arcs,

flaming arcs, magnetic arcs and Mazda lamps, it was found that these last made a very favorable showing. The intensity of illumination varies somewhat in the different departments and is about as follows: 4 ft.-candles per sq.ft. in machine, woodworking and erecting shops; 3 ft.-candles in forge and repair shops; 6 ft.-candles in paint shop, and 1 ft.-candles in storeroom. Glass reflectors and steel reflectors of different types are used.

Steel versus Creosoted Wood for River Barges*

By A. E. Hageboeck†

There is a great deal of discussion at this time regarding the economy of steel barges as compared to the wooden craft. When the wood is not treated, it is probably true that steel is the better material, but in the writer's opinion the steel is no more economical than the creosoted wood if one considers the additional investment required in the case of steel barges. To show this point, figures obtained from actual records in the U. S. Engineer Office at New Orleans will be introduced as evidence. This office has been collecting data on the cost of repairs of the standard barge, 100 ft. x 20 ft. x 4 ft. 7 in., for the past 20 years. As we are building the same size barge today, the costs of repairs will be directly comparable.

In the past five years the writer has examined a large number of untreated-wood barges that have been in service on the Mississippi River from four to fifteen years. From these observations he would say that the decay always starts where there is an excess of moisture in combination with air. In 90% of the cases the decay starts in the ends of the timbers. That is to say, the decay develops in the same ratio as the wood absorbs moisture through the ends. Since a good pressure treatment will always plug the ends of the timbers, it is easy to understand why such good results have been obtained with a pressure treatment of coal-tar creosote oil.

In former years the opinion was held that it would not pay to creosote a barge because it would wear out before it decayed. This may be true under certain conditions, but as a general proposition it has been found that the lumber decays first and when in this decayed condition is easily broken up. For barges used in fresh water it is not considered necessary to creosote the bottom, as it has been found that the bottom planking rarely decays. This can probably be attributed to the exclusion of air by the 4 to 6 in. of water usually found inside the barge. It has been found practical to frame the timbers, creosote them in transit at a commercial treating plant, and then forward the timbers to the point of erection. By marking the pieces that cannot be otherwise easily identified, it is possible to assemble the barge quite rapidly.

In constructing light-draft barges it has been our policy to use the pressure-creosoted fir, as fir can be obtained in long lengths at a reasonable cost. Long timbers are especially desirable in barge construction because they reduce the number of gunwale joints. The gunwale joints are always the first points to cause trouble by leaking. Besides being cheaper in cost, both before and after creosoting, the fir is lighter and gives a draft of but 9 in. for a standard barge 100 ft. x 20 ft. x 4 ft. 7 in.

In the case of boat-shaped steamboat hulls, creosoted timber is eliminated from consideration on account of the necessity of framing and cutting timbers during erection, which would expose untreated surfaces if creosoted timbers were used. This condition, however, does not exist in the case of scow-shaped boats and barges.

A steel barge will cost more than three times as much as an untreated-fir barge and nearly three times as much as creosoted barge. The lumber for these scow-shaped boats and barges can be advantageously framed and bored before treatment.

TABLE I. COMPARATIVE COST OF TREATED AND UNTREATED YELLOW PINE BARGES 120 FT.x30 FT.x6 FT. 0 IN.

Items	Untreated Barges 10 yrs. old	Creosoted Barges 9 yrs. old
Original Cost	$3093.39	$4000.00
Cost of Repairs	2006.61	557.35
Total Cost	$5100.00	$4557.35
Subtracting value of Barge today		3600.00
Total Cost of Barge	$5100.00	$ 957.35
Annual Cost per Barge	$ 510.00	$ 106.00
Annual Saving in favor of Creosoted Barge		$ 404.00

TABLE II. COMPARATIVE COSTS OF LIGHT-DRAFT BARGES OF VARIOUS MATERIALS,

SIZE 100 ft.x20 ft.x4 ft. 7 in.

Items	Douglas Fir Untreated 15 yrs Life	Douglas Fir Treated 10 lbs. 20 yrs. Life	Yellow Pine Untreated 15 yrs. Life	Yellow Pine Treated 14 lbs. 22 yrs. Life	Steel 25 Yrs. Life
Original Cost	$1200	$1500	$1300	$1650	$4000
Total Repairs	1094	400	1094	700	400
Interest at 5% on Cost	900	1500	975	1815	5000
Interest at 5% on Repairs	341	125	341	125	125
Total Cost	$3535	$3525	$3710	$3990	$9525
Annual Cost per Barge	$ 236	$ 177	$ 247	$ 182	$ 381
Annual Saving in favor of Creosote Fir Barge	$ 59	.....	$ 70	$ 5	$ 204

The creosoted barges have been appraised at $3600 for the reason that they are in a perfect state of preservation and serviceable for at least 10 years more.

Yellow-pine Barges

On the Mississippi River, between St. Paul and St. Louis, untreated yellow pine has been used but little, and the writer has been unable to obtain any accurate records of its lasting qualities. It is generally believed that unless timber practically free from sap is obtained its life would be exceedingly short. On the lower Mississippi River a yellow-pine untreated barge containing a minimum proportion of sappy timber is past economical repairs at the end of 10 years.

Pressure-treated yellow-pine barges have been used on the lower river for 12 years. These barges are today in a perfect state of preservation and without doubt are good for an additional life of 10 years. It has been found necessary to re-calk the barges after two years' service, but otherwise the repairs have been small and but little further re-calking seems necessary during the life of the barge. The lumber in these barges was treated while in a green condition and, in the opinion of the writer, the real necessity for re-calking was due to the subsequent shrinkage of the timber and consequent opening of the seams and loosening of the oakum.

On the lower Mississippi River, where there is always a good stage of water, light draft is not a controlling factor, and so barges 120 ft. x 30 ft. x 6 ft. and larger are in general use. The original cost of these untreated yellow-pine barges built in the early '90's was about $3000, and the cost of repairs during the life of 10 years averaged $2007 per barge. The original cost of similar barges built of pressure-creosoted yellow pine was $4000 and the cost of repairs on 10 barges averaged $557. The total cost of the untreated barge was $5100, and nothing remained at the end of 10 years to show for this expenditure. The total cost of a creosoted barge during nine years was $4557 and a good barge now appraised at $3600 remains on hand. Table I shows the comparative annual costs per barge.

Untreated Douglas-fir Barges

With but few exceptions the necessity for repairs to an untreated barge is due to decay and not to mechanical abrasion. Ordinarily the decks of barges used for rock transportation will first decay on the bottom side at the points of crossing other timbers, and in this weakened condition are easily broken. Deck planks which originally were 2½ in. in thickness, after eight years' service still measured 2⅜ in., but because of their decayed condition, and not because of wear, it was necessary to replace them. It is not uncommon to find evidence of decay on untreated-fir barges after three years' service.

The records of the U. S. Engineer Office at Rock Island, Ill., of the repair cost on Douglas-fir barges in use for the

*Abstract of an article read at the Chicago meeting, 1912, of the American Wood Preservers' Association.

†Inspector in charge of creosoting operation, U. S. Engineer Office, Rock Island, Ill.

past 20 years in connection with the work of improving the upper Mississippi River between St. Paul and St. Louis may be summarized as follows: During the sixth and seventh years the barges required extensive repairs, the cost ranging from $200 to $300 per barge; with repairs costing about $75 per year they continued in hard service to the tenth or twelfth year; they then required large repairs and had to be taken from rock work and placed in the brush-carrying service, which is much less severe; from this time on to the end the cost of repairs per barge was largely increased, and it is debatable whether it would not have been fully as economical to abandon the barges at about the tenth to twelfth year instead of keeping them in service up to 17 years, as was often done.

Treated Douglas-fir Barges

It seems safe to estimate the life of creosoted-fir barges at 20 years, since untreated barges have given an average life of 15 years. The major portion of the repairs on an untreated barge is for calking and repairs to deck, rake and gunwale joints on account of decay. As the present tendency is to air-season the fir before treatment, it seems natural to believe that the barges will give a long service without re-calking. As an additional precaution it is thought advisable to protect the creosoted deck with a 1-in. wearing service of untreated material. The repairs to the deck are, therefore, confined to the occasional relaying of this protection.

Table II, for timber barges, is based on government freight rates on timber. Therefore, for commercial comparison, $10 per barge should be added to the yearly cost.

The yearly costs in Table II show the following relative order for economical barge construction: (1) Creosoted fir, (2) creosoted yellow pine, (3) untreated fir, (4) untreated yellow pine, (5) steel. It will be noted that the annual cost of a steel barge is twice that of either a fir or a pine creosoted barge.

In conclusion the writer will state that the excess of original cost of a steel barge with interest on the investment is not compensated by the added life as compared with a creosoted barge. The cost of repairs on an untreated barge and its short life of real hard service makes the annual cost, including interest, 25% most than for the pressure-creosoted barge. The pressure-creosoted yellow-pine barge and the pressure-creosoted Douglas-fir barge have respective fields, depending upon the working conditions; on the lower Mississippi, where there is always a good stage of water, the creosoted yellow pine will probably be more desirable, but for light-draft use on the upper Mississippi the pressure-creosoted fir will be far more economical.

The Adjustment of the Expert to City Government*

By Thomas H. Reed†

The development of city government in the United States during the past 10 years has been swift, indeed. We have followed the example of stricken Galveston and established simple city governments in place of the complex forms which so long perplexed good citizenship. Simplicity has brought with it a great improvement of the actual conditions of city government. "Economy" and "efficiency" have been restored to the municipal vocabulary.

There remains, however, one further step to take. Great advance on its predecessors in the course of our civic evolution, as commission government has proved, it has affected but one of the conditions on which good city government depends. It has created effective political responsibility on the part of the elected officers of the city. Its good results are sufficiently explained by this fact. It has, as yet, scarcely touched the all-important relation of the politically responsible officer to his subordinates. A few commission-governed cities have a system of competitive examinations as a basis for appointments in their service. These merit systems mark no advance on the typical American brand of civil service reform. They are applied to only the lower and more poorly paid positions, while the better paid and more responsible positions are open to any one, fit or not. In the larger commission-governed cities the members of the council are paid fairly large salaries and expected to give all their time to the work of the city. In the smaller places where smaller salaries are paid, part time only is demanded. In all the cities, large or small, which have adopted commission charters, save Galveston alone, the councilors are expected to fully run their departments with the assistance of comparatively poorly paid subordinates. In other words, the relation of the political department of the city government to what, at least, should be its nonpolitical departments, is the same in the Berkeley of 1912 as it was in the Berkeley of 1908.

That the actual work of city administration should be carried on by competently trained men is a principle more frequently recognized in its abstract form than in its concrete applications. Water supply, street cleaning, public works, police protection are functions in these days for the expert. No other undertakings of similar magnitude are ever intrusted to amateurs. Yet we put the actual work of city administration on men selected by popular election. To hold that "government" is a task within the range of every intelligent citizen, has been the greatest self-deception of American democracy. Such a view, however, while it is false, is at least comprehensible and consistent. But to believe in government by experts and yet look to popular election under the commission plan to select and maintain in office such experts, is mere midsummer madness. A recent writer on commission government prophesies that the council will become a permanent board of administrative experts similar to those which govern German cities. It is a childish dream.

The permanent board of administrative experts—the magistrat—of a German city consists in part of paid, in part of unpaid members, selected for long terms by the city council. The permanent officials of an English city, whose training and ability have been largely responsible for the municipal success in that country, are selected by the city council. Both in England and Germany it is the custom to advertise for candidates and to appoint men whose experience in a smaller city offers promise of success in a larger field. The very nature of a popular election precludes the possibility of such a search for officers. The electorate cannot advertise, it must simply say yes or no to those who propose themselves to it. No expert worth his salt will subject his career to the perils of a popular election. There must be an intermediary of some sort between expert and people, able to appreciate the services of the expert and provide him some protection from popular prejudice, and at the same time competent to scrutinize intelligently his activities. Some mitigation of the red-tapely rigors of officialdom, a potential control in the interests of the people of the expert's conduct, are the chief functions of the elected representatives of a city democracy. It is their business to choose men of character and capacity to be the working heads of each department; to supervise their conduct; to uphold them while worthy of confidence; to dismiss them when confidence is no longer possible.

It is this nice adjustment of expert and politician which has made English city government at once popular and efficient. We, even in our most progressive type of city government, have been blind not only to its importance, but to the very thing itself. We are not politically conscious of the necessity of expert service. We recognize the expert in law, medicine or business, but depend upon the ubiquitous genius of the American people for the far more complicated tasks of city administration. The establishment of the proper relation of council and expert is the only way by which we can secure first rate service for our cities.

The next step forward is the proper adjustment of the expert to our otherwise admirable system of city government.

*From an adress before the Berkeley City Club, as presented in "The California Outlook," of Los Angeles.

†Associate Professor of Government, University of California, Berkeley, Calif.

Standard Practice Cards for Maintenance of Way; Erie R. R.

Instructions regarding work for maintenance-of-way and structures are issued by most railways in books of convenient size, which include also general rules and regulations. With this method, however, it is difficult to revise the rules or to supplement the book by changes, additions or special instructions. On the Erie R.R. the book method has been replaced by the card method, in which the rules or instructions relating to different matters are printed on cards of uniform size which can be fitted to a cover or case for the pocket. In this way new or special instructions can be issued as required, and old ones canceled by an order directing the removal of cards of given number. For information concerning this system we are indebted to J. C. Stuart, Vice-President, and to C. H. Moore, Principal Assistant Engineer.

The cards are 6x4 in., printed on paper which is strong and tough, but not very thick. Each one is numbered and dated and bears the name of the railway and the department, and the title of the instructions with the heading "standard practice." The face of one of the cards is shown in the accompanying cut. No two kinds of work are included on one card, but two or more cards may be used for one set of instructions that is too lengthy for a single card. They are printed on both sides if necessary. Each card is punched with two holes in the lower 6-in. side, so that it can be fitted to the clips or spring staples of the loose-leaf cover. This cover is of limp buff leather 6¼x5 in., with the back wide enough to take a pack of the cards about ¼ in. thick. Cards can be inserted at any place, so as to maintain the proper sequence or numbering. The first cards contain an index, which is printed across the narrower width. The others are printed across the 6-in. length, and in such a way that the filing holes are at the bottom of the front and the top of the back of the card. In this way the printing all reads the same way, as the cards are turned over. Some of the "standard practice" cards already issued are as follows: 1, Elevation and gage on curves. 2, Standard roadbed sections. 3, Ballasting. 4, Laying rails. 5, Program for season's work for section forces. 6, Distributing and piling ties. 7, Applying tie plates. 8, Switch heating equipment. 9, Stringers for wooden spans. We give below the instructions on certain subjects, slightly modified from the original.

BALLASTING

METHOD—In stone ballast, pick tamp all ties on both sides from the ends to a point 12 in. inside rail. Center of ties to be lightly filled by use of ballast fork. Allow sufficient time (minimum, one week) to elapse for ties to secure a good bearing, then resurface the tracks with a light raise. In resurfacing joint ties they should be tamped hard at the joint end, easing off on the quarter.

When track has been put to a true line, fill and trim to standard section. Ballast should be neatly lined on the outside by using a straight-edge 6 in. wide. All surplus ballast should be piled and promptly removed after second raising and resurfacing. Use ballast forks for handling stone ballast.

In gravel, granulated slag and cinder ballast, tamping picks should be used only on joint ties, using shovels on all others.

FORCE (STONE BALLAST)

Foreman	1
Asst. foreman; sighting track	1
Asst. foreman; with track tampers	1
Laborers; digging jack holes	1
Laborers; holding ties tight to rail	2
Laborers; driving spikes home	2
Laborers; operating jacks	4
Laborers; tamping at jacks	4
Laborers; tamping	16
Laborers; partially filling and dressing track	4
Water carrier	1
Total	37

LAYING RAILS

Location is determined upon at the end of the calendar year. During the winter, when track work is impracticable, the adzing should be done, as far as possible. Whenever possible the Supervisor should be present when the rail is relaid.

FORCE (FOR LAYING SINGLE RAIL)

Foreman	1
Asst. foreman	1
Asst. foreman	1
Flagmen	2
Tong men	8
Applying joints	8
Water carrier	1
Spike pullers	4
Spikers	4
Adzers	5
Shim men	2
Tools and supplies	1
Rail removers	2
Total	40

When two rails are laid, the above force (excepting the flagmen) should be duplicated. When two lines of rail are relaid, the second rail should not be laid until the first rail is in place and spiked. On straight track it is permissible to couple rails together before sliding into exact position. On curves, rails should be laid one at a time. Idle time should be used in removing angle bars from old rail.

Spike pullers pull all spikes to permit the old rails to be removed. When the proper gage can be maintained, draw spikes on outside of rail. When rail wear or change in design of rail affects the gage, pull spikes on outside of one rail and inside the other; and where necessary, pull the two outside lines and one inside line of spikes. The proper gage to be made as the new rail is laid. In no case shall rail be laid to a closer gage than authorized in standard practice.

Rail removers carry lining bars to throw out old rail. Adzers do all necessary adzing. Tong men place new rail into position. One of the shim men removes shims and clears rail seat; the other places and adjusts shims by use of thermometer. The number of joint men given is for applying new joints. When continuous joints are used, two additional men should be used. The spikers spike new rail in position and carry a gage to correct bad spots immediately. The man in charge of tools and supply collects (with a light push car) all tools, spikes, bolts, nuts, nut-locks and any small material left on track.

TIE INSPECTION AND RENEWALS

INSPECTION—The selection of ties to be removed from track for renewal on each division should be made by one or more track-tie inspectors, as soon after Mar. 1 of each year as weather conditions will permit, and prior to July 1 will submit his report direct to the division engineer.

Each tie to be removed must be carefully examined, the local conditions being carefully studied and taken into consideration; the condition of ties on each side, and the amount and character of traffic, the location of the ties, whether on a tangent, curved track, or under a joint, etc., must be carefully considered. A much more rigid inspection is possible on straight track with light traffic, than where traffic is heavy and on a sharp curve.

When the inspection is made the section foreman on each section shall be present and assist in the work. A third man will be necessary to do the spotting and carry the work along rapidly. The inspection should begin at the lowest mile post and work toward the greater.

MARKING—Ties to be removed should be designated by marking the web of the rail directly over the tie, with a spot of white lead paint, as directed by the engineer of maintenance of way. Where it is expected the rail will be removed prior to the change of ties, in addition to the spot on the rail, the tie should be spotted near its end.

LOCATION—As the tie inspection is made the total number of ties in each mile will be counted and shown, and the number to be renewed in each quarter of a mile will be shown; also the number of unapplied ties. Where there is no mile post, telegraph poles will be used to designate the location.

No. 7. **ERIE RAILROAD COMPANY.** 3-31-11

NEW YORK, SUSQUEHANNA & WESTERN RAILROAD — NEW JERSEY & NEW YORK RAILROAD — CHICAGO & ERIE RAILROAD

MAINTENANCE OF WAY AND STRUCTURES

STANDARD PRACTICE: Ties. Distributing and Piling of New Ties.

Ties will be received for the following year's renewals after working season is closed. They should be held on hand in cars until a sufficient number are received to justify the use of an engine to unload. Then distribute and place as near to point of using as possible, unloading from 40 to 50 ties to the pile.

Ties should be piled pyramid or "A" shape not higher than 6 feet, lightly covering pile with cinders or earth and removing all dead grass and weeds from around them. Piles of ties to be not less than 50 feet apart.

In carrying ties to piles men are to work in sets of three, two walking ahead with wooden bar under tie and one at rear.

In unloading ties from cars men are to work in sets of three and not to exceed nine men on car.

In loading ties men are to work in sets of five, three men carrying to car and two on car.

The number of men carrying ties should not exceed three at any time.

SAMPLE OF "STANDARD PRACTICE" CARDS FOR MAINTENANCE OF WAY AND STRUCTURES; ERIE R.R

(Cards 6x4 in., for loose-leaf binder.)

When the inspector has finished each section, he will forward one copy of his inspection report to the division engineer, one to the supervisor and one to the engineer of maintenance of way, and leave one with the section foreman. The supervisor will then be enabled to distribute the ties needed for each mile or quarter of mile.

RENEWALS—When the section foreman has completely finished the renewal of ties on any one mile, he will forward a statement to the supervisor, showing the exact number of ties that he changed on the section, whether it agrees with the inspection report or not (and if not, an explanation why). The supervisor will then forward to the division engineer and the engineer of maintenance of way a report showing the number of miles completely finished on his subdivision, on the 10th, 20th, and end of each month. No ties should be reported in a mile unless all the ties to be changed have been put in. No ties shall be removed from the track, except broken ties or ties marked for renewals (except in cases of emergency), without the express permission of the division engineer to vary from the inspection.

SWITCH TIES—Switch ties which have become unserviceable from decay or other causes will be renewed. These will be measured and the number and length of each to be renewed will be recorded and the rail painted by the inspector, as for cross-ties. The track and switch should be named and this information given the division engineer to enable the proper distribution of the switch ties to be made from the cars, which should be loaded in station order and lengths for this purpose.

SIDINGS—It is desirable that as far as possible the above instructions be applied to renewal of ties in sidings. The record of ties should be kept with the name of the sidings instead of the quarter-mile, the painting or marking to be done in the same way. Chestnut and cedar ties may be used on tangents of standing or back-off tracks. A separate list should be kept covering such tracks.

GENERAL—It should be distinctly understood by the track supervisor that the above standard practice does not in any way relieve him of his responsibility for the proper maintenance of the track under his charge.

APPLYING TIE-PLATES

PREPARING TRACK FOR ADZING TIES—It is required that the ties be first scored, scribed or cut crosswise with the adz, both outside and inside the rail, and as far back from the rail as the adzing is to extend to suit the plate to be used, cutting the ties as deep as the adzing is likely to be required. Pull both inner and outer spikes of rail only for a distance sufficient to admit of rail being sprung or moved toward center of track, clear of its bed, to give room for the proper adzing of the ties.

ADZING TIES—It is required that all spike holes be plugged. Then adze the ties level the entire width to a uniform surface, to extend not less than ½ in. beyond the ends of the plates. To insure a true and even surface, and prevent it being concave or hollowed out, the adzing should be diagonally and across the tie. The adzing of low rail of curves should be done so as to cant the rail inward, not less than ⅛ in.

APPLICATION AND SPIKING—Tie plates should now be placed on the beds prepared for them and the rail moved back to its position on top of the plates. No plates should be imbedded until at least every second plate on curves and every third plate on tangents has been spiked to place, using the gage on opposite rail (yet undisturbed) to bring rail to gage. In spiking to plates, spikes should be driven plumb, care being taken to bring the shoulder of plate snugly up to base of rail for its entire length.

When plates are applied on curves, the plates should be driven a little so as to give them a hold on the tie and thus aid the spikes holding the rails to place until they are fully imbedded and spikes driven home. In driving the plates down, it is required that both ends of plate be struck at the same time. When driving plates down on low rail of curve, the inner end of plate should be driven further down than the outer end, as the superelevation of the curves throws the greater weight toward the outer base of rail, thus settling that end of plate quicker than the end on the inside of track. One rail of track being thus equipped with plates the work may now be repeated on the opposite rail, thus having always one rail to gage from.

Fire Tests of Partitions for Buildings

The Division of Buildings, of the Department of Public Safety, Cleveland, Ohio, has conducted a series of tests of different types of partitions for buildings (at the request of the Lathing Contractors Association), testing their resistance to fire and to the subsequent action of streams of water from fire hose. The panels were 10x8 ft., built into steel frames hinged so as to be swung against an opening in a furnace fired by oil. The fire test was for two hours, after which the panel was swung back and a stream of water was thrown on it for one minute from a hose connected to a city hydrant and having a 1⅛-in. nozzle. The tests were made under the direction of a committee appointed by V. D. Allen, Inspector of Buildings. The committee was composed of L. H. Miller, an engineer of the Bethlehem Steel Co.; Prof. Nelson, of the Case School of Applied Science, and Wm. S. Lougee, architect. The Division of Buildings was represented by A. W. Zesiger, Engineer of Construction, who has checked for us the accompanying abstract of the results.

The panels were allowed to set for two weeks before being tested. Their construction and the results of the tests are described below:

PANEL No. 1—This was of No. 24 painted expanded metal lath stapled to 2x4-in. pine studs, 12 in. c. to c., plastered with a cement and lime plaster on ¾-in. grounds, about 5½ in. thick over all. It took 30 minutes to bring the temperature up to 1700° F., and for the remaining 1½ hours it ranged between 1700° and 2000°, the maximum reached being 1912°. At the end of the two hours, the door was thrown back and water from a city hydrant was directed against the hot panel through a 1⅛-in. nozzle within one minute after the door was opened. After cooling, the metal lath and plaster was torn down to see what had been the effect on the wood studs and outside of wall. What was left of the studding was charcoal, but the outside of the wall was intact and in a condition to resist more fire.

PANEL No. 2—This was of white pine lath on wood studding, 16 in. c. to c., plastered with a patent gypsum plaster on ¾-in. grounds. The fire reached a maximum temperature of 1865°, but in less than an hour all but the outside shell of plaster was destroyed and that was gradually cracking and opening up; this allowed the cold air to enter the chamber, with the result that it held together long enough to give it the two hours' fire. After the water had been thrown on it, there was practically nothing left of the panel.

PANEL No. 3—This had painted No. 24 expanded metal lath wired to ¾-in. steel channel studs spaced 12 in. c. to c., plastered on both sides with lime and cement mixture, making a solid wall 2 in. thick. The grounds were ¾ in. thick on the lath side and ½ in. on the other side. On this partition the maximum temperature was 1929°. When water was thrown upon it, with the exception of the washing off of a thin layer of the plaster (that had recalcined under the intense heat) the partition had the appearance of being able to go through another such test.

PANEL No. 4—This was a stucco wall such as would be built with metal lath and cement plaster for the outside wall of a house. The metal lath was fastened direct to the 2x4-in. pine studding, then plastered and back-plastered between the studs, giving a thickness of 1½ in. This side, corresponding to the outside of a house, was placed next to the fire, as it was desired to learn how far this construction might prevent the spread of a conflagration through a residential district. On the outer side of the studding, which would correspond to the inner side of the wall, No. 24 metal lath was fastened and plastered the same as panel No. 1.

The mixture of the cement plaster was particularly designed to prevent hair cracks and other imperfections to which stucco walls under alternating weather conditions are subject when not properly built. It seemed to stand the abuse about as well as, if not better than, panel No. 1. The furnace was fired the full two hours, the highest temperature reached being 1943°, and water was then poured upon it. After the test the fire side of panel (representing the exterior of a house) was intact; on tearing off this side after cooling, the wood studding was found in much better condition than in panel No. 1.

PANEL No. 5—This was made by wiring metal lath on both sides of a studding 2½ in. over all, built by fastening two ¾-in. steel channels together. The cement plaster was applied to both sides alike, as in panel No. 1, with ¾-in. grounds, thus making a 4-in. hollow metal lath partition. The temperature reached 1976°. The metal lath was slightly exposed on the inside of the wall, but it was thought the test might have been repeated before the partition would be destroyed, as the outside of the wall had received no damage and the fire side was good for service to protect the outside from heat.

PANEL No. 6—This was ⅜-in. plaster board nailed to 2x4-in. pine studding. The plaster was put on ¾-in. grounds in three coats. This panel had a total fire test of 74 min. at a maximum temperature of 1562°. Then water was allowed to flow on it at low pressure to quench the fire in the panel, after which the full stream was turned on for half a minute, with the result that practically nothing was left of the panel.

Recent Practice in Diamond Drilling and Borehole Surveying*

By John I. Hoffman†

Until comparatively recently, boreholes were put down without any special care being taken in trying to keep them vertical, and consequently some of the boreholes which were sunk on the Rand, South Africa, never intersected the reef at all, although it was a known fact that the reef was there, the boreholes having deviated to such an extent that they followed a course more or less parallel to the strike of the reef at some distance above it.

Recently, however, such penalty clauses have been inserted in the diamond-drilling agreements, that the greatest care has been exercised by the contractors to keep the holes vertical, and consequently many devices and contrivances have been employed to accomplish this desired result.

The surveying instrument which is now exclusively used on the Rand, was invented by Oehman‡ and improved upon by A. Payne-Gallwey (Fig. 1). The instrument is an electric-light photographic apparatus and consists essentially of a gun-metal tube in two halves *A* connected by a coupling *O*. In the lower half of the gun-metal tube are placed a magnetic needle *B* and a plumb-bob *C*, each independent of the other and each swung over a gimbal *D*. Above the needle and plumb-bob, respectively, is fixed a small electric lamp *E*, and all are held in position and pressed against an insulated brass rod *F* in the center of the coupling, by a spiral spring *G* attached to the bottom screwed plug *H*. In the side of the tube, a series of small screws *I* are placed in a straight line parallel to the side, their ends projecting inside the tube about $\frac{1}{16}$ in. The cylindrical cases carrying the lamps and those carrying the needle and plumb-bob have a slot down the side, the projecting screws acting as guides for the slotted cases to slide into and keep them in position.

The top half of the tube contains a dry battery *K* and a clock *J* which has a spiral spring *L* attached to it. The spring presses against the top end-piece *M* of the tube so that when the two halves of the tube are screwed together everything inside is held rigidly together and contact assured by means of the spiral springs at top and bottom. To the top end-piece a ball-bearing swivel *N* is attached in order to lower the instrument on a wire, if necessary.

The cases *P* which carry the marine-compass attachments for the magnetic needle and the plumb-bob are made of vulcanite, for insulating purposes, the compass attachments being made of brass, the outer ring of which is held in position by two brass screws on which the ring swings. On the face of each gimbal is a fixed pin point and round the edge is a recessed ring which holds the disk of sensitized photographic paper in place, the pin points holding them in position. The plumb-bob is made of gold attached to a fine silk thread swung from the center of a thin disk of plate glass *Q* which fits into a recess in the top of the vulcanite case. Both the magnetic needle and the plumb-bob swing immediately above (almost touching) the sensitized papers.

The watch has an extra wheel *R* to which is attached a copper projection *S* which at a certain set time makes connection with a copper spring *T*, attached to the frame of the watch, and completes an electric circuit, lighting up the lamps above the plumb-bob and the needle and photographing a sharp shadow of each on the sensitized paper. When the two photographs are developed, the dip and direction can be read off by making the pin pricks coincide.

The Deepest Boreholes in the World—I will now describe in detail two of the most important holes which have recently been sunk, both successfully, on which the latest inventions were used. The two holes in question are (1) the hole put down by the Lace Proprietary Mines on the farm Vlakfontein in No. 26 in the Heidelberg District of the Transvaal, and (2) the hole put down to the south of the Brakpan Mines on the portion of Witpoort No. 162 belonging to the South African Lands and Exploration Co., Ltd. Both these holes were put down by the same contractor, William Gallagher, of Johannesburg. I take this opportunity of mentioning his name because in both cases he created a world's record.

The precautions taken in both these holes to try to keep them plumb were as follows: Special core-barrels were made having an outside diameter exactly the same as the outside diameter of the crown, and these core barrels were made 30 ft. long instead of 10 ft. as usual, so that there was practically a rigid length of rod of 30 ft. 5 in. from the cutting face of the crown to where the core-barrel joins the rods. As a rule the core-barrel is only a shade larger than the rods, which, in the case of a Sullivan "B" Drill, measure $1\frac{7}{8}$ in. outside diameter, whereas the outside diameter of the crown measures from 2 to $2\frac{1}{16}$ in. before the diamonds are set into it. After the diamonds are set in the crown the actual cutting diameter measures about $2\frac{1}{8}$ in.

It will therefore be seen that the ordinary rods have a diameter of ¼ in. less than the cutting diameter of the crown, and consequently immediately above the crown, which is only ¾ in. deep, the core-barrel and rods can cant almost ⅛ in., until they touch the side of the hole. It is therefore obvious that, once any pressure is put on the crown, the flexibility of the rods will allow them to cant up against the side of the hole, and consequently there will be more pressure on the crown on the side of the hole to which the rods have canted than the opposite side, which is the cause of the drill commencing to deviate from the vertical.

As in the case of the two holes mentioned above, where special core-barrels were made having the same diameter as the crown, there was only sufficient clearance between the outside of the core-barrel and the borehole to allow the water and the tailings to flow past, and this had the desired effect of keeping the hole absolutely plumb in the Lace Proprietary Mines Vlakfontein borehole for a depth of 1000 ft., after which the drill commenced to deviate, probably owing to the bit entering broken strata.

As I have already mentioned, these two boreholes have constituted a world's record. In the case of the Lace Proprietary Mines' Vlakfontein borehole, it reached a depth of 6656 ft. (measured along the rods), the deepest borehole in the world. Once the hole left the vertical at a depth of 1000 ft., the angle of deviation continued to increase gradually until, at the bottom, the hole was dipping at 58° from the vertical.

From 1000 ft. to 2600 ft. the rods deflected gradually from the vertical to 44°, but continued at more or less that angle of dip down to 5000 ft., where the deflection again commenced to increase, until at 6600 ft. (the lowest depth surveyed) the dip of the hole was 58° from the vertical. The reef was struck at 6198 ft. (measured along the rods), the calculated vertical depth being 4813 ft. The horizontal distance between the points where the reef was struck and the borehole commenced was 3259 ft.

New Cores from the Same Borehole—Owing to the great expense of sinking these deep boreholes and only getting one section of the reef, an idea of Mr. Welldon's (at that time foreman driller on the Apex borehole) was improved on, and carried out by A. Payne-Gallwey, of Johannesburg, whereby another section of the reef could be taken from the same borehole, in any required direction, by means of a deflecting wedge.

This contrivance consists of a pilot wedge, a guide wedge and a main deflecting wedge, and the method of procedure is as follows: In the case of a Sullivan "B" hole ($2\frac{1}{8}$ in. diam.) a

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*Abstract of a paper read before the Institution of Mining and Metallurgy, London, Eng., Apr. 18, 1912.

†216 Moorgate Station Chambers, London, E. C.

‡This instrument was described in a paper read by Dr. F. H. Hatch before the British Association in 1905, entitled "An Instrument for Surveying Deep Boreholes." (British Association Report, 1905.)

pilot wedge (Fig. 2) is made of round iron, 2 in. in diameter, the length being 18 in. over all and the wedge face being about 6 in. long. The end opposite the wedge face is threaded to fit a piece of 1½-in. inside diameter piping. A ¾-in. hole is drilled in the face of the wedge and tapped, and a nick *A* is made with a chisel on the top of the wedge, along its longitudinal axis. This nick is for surveying purposes, as will be explained later on. This wedge is screwed into the piece of 1½-in. piping about 3 ft. long, with teeth cut in the end opposite to where the wedge is screwed into it.

The borehole is then filled with water and the pilot wedge and piping are dropped down the hole, wedge face upward. The main rods are then lowered to find out whether the pilot wedge and pipe are resting on the bottom of the hole. If not, the wedge has to be lifted, by means of the tapped hole referred to above, and again dropped, until it is found that the wedge and pipe are resting on a solid bottom, or at any rate are fast and rigid somewhere near the bottom of the hole.

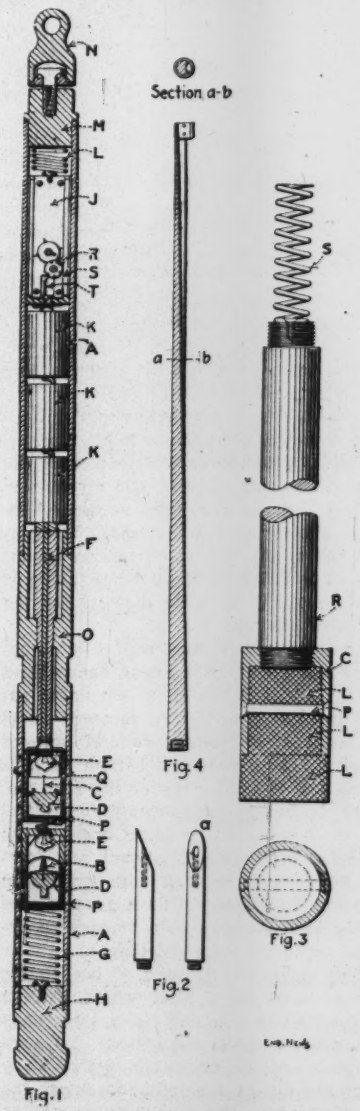
The position of the wedge is surveyed with the instrument shown in Fig. 3, which consists, for a "B" hole, of a brass rod *R* about 1⅛ in. in diameter and 3 ft. long, threaded at both ends. To one end is fixed a spiral spring *S* similar to that fixed to the bottom plug of instrument Fig. 1. To the other end a brass cup *C*, 2 in. in diameter, is screwed, having a ¼-in. diameter brass pin *P* riveted through across the diameter of the cup. This cup is filled with lead *L* which projects about 1 in. beyond the edge of the cup and which is turned to the same diameter (2 in.). The end of the rod with the spiral spring is screwed into the bottom of the instrument, previously described (Fig. 1), in place of the bottom plug.

The relative position of the pin-points on the gimbals to the guide inside the case of the instrument being known, the position is marked on the outside of the case and the line continued along the brass rod, cup and projecting lead. The top end of the instrument is screwed into a brass tube 10 ft. long and that again screwed on to the end of the drill rods.

The whole arrangement is lowered down the hole until the lead is resting on the top of the wedge, which, with its chisel cut, makes an impression on the lead, a photo of the magnetic needle being taken at the same time. On raising the rods a disc of lead with the impression is sawed off and the direction of the wedge calculated.

The guide wedge is an exact counterpart of the pilot wedge, and the butt end of this is screwed into the butt end or bottom of the main deflecting wedge.

The main deflecting wedge (Fig. 4) is made out of a solid piece of 2-in. diameter round steel about 7 ft. long. The bottom end is drilled and tapped to receive the butt end of the guide wedge. The top end is drilled with a 1¾-in. bit, leaving a 2-in. ring of metal ⅛ in. thick. The solid portion of the 7-ft. length is then planed, commencing from the underside of the metal ring down to about 1 ft. from the bottom, in such a manner that a wedge is formed with a concave



INSTRUMENT USED FOR DETERMINING DIRECTION OF BOREHOLES FOR DEFLECTING DRILL RODS TO OBTAIN NEW CORES FROM SAME HOLE

face with a radius of 1 in., and having a thickness of ⅛ in. at the top and 2 in. at the bottom end (Fig. 4).

For surveying purposes a chisel cut is made in the top of the ring, along its diameter, coincident with the longitudinal center line of the wedge, as shown in section *a—b*, Fig. 4.

The direction of the face of the pilot wedge now being known from the survey, the guide wedge is screwed into the main deflecting wedge in such a way that when the faces of the pilot and guide wedges are together, the main deflecting wedge is facing in any predetermined direction.

The end of a "C" rod (1⅝ in. diameter) is now placed in the metal ring at the top of the main deflecting wedge and riveted thereto with two ¼-in. copper rivets. To the other end of the "C" rod is attached a 10-ft. length of "B" rod (1⅛ in. diameter), on the top end of which is screwed a ball-bearing swivel, and the main drill rods are connected to the top of the swivel.

The whole contrivance is then lowered down the hole, and, as soon as the point of the guide wedge touches the face of the pilot wedge, the swivel allows the guide wedge, and with it the deflecting wedge, to revolve until the faces of the guide and pilot wedges are coincident, the top one sliding on the bottom one until they jamb. If the weight of the rods is not sufficient to shear the rivets, hydraulic pressure is put on the piston of the drilling machine and the copper rivets sheared off, leaving the main deflecting wedge in position at the bottom of the hole.

The main rods, together with the "C" rod which was riveted to the top of the main deflecting wedge, are now withdrawn from the hole, and the direction of the main deflecting wedge is surveyed to ascertain if it is in the required position.

If the position of the main deflecting wedge is correct, "C" rods with a "C" bit are now attached to the main rods and these are lowered down the hole, entering the ring at the top of the main deflecting wedge which acts as a guide. Drilling is commenced with the "C" bit and continued along the face of the main deflecting wedge, which throws the bit over into the side of the previous borehole.

Up to the beginning of 1910 this "C" hole would have had to be continued down to the desired depth, the main deflecting wedge being left in position in the hole until drilling was completed so that the top ring of it would always act as a guide for the "C" rods to enter the new hole, and consequently no further deflection could be made from the "C" hole, as it would have meant again decreasing the size of the hole.

In April, 1910, it was decided to try to obtain another core of the reef at the Lace Proprietary Mines borehole, commencing the deflection about 800 ft. above the bottom of the hole, and this work was carried out successfully as follows: As the hole had been standing for five months a considerable amount of sediment had settled in the bottom and this had to be cleared out first of all, which was done by drilling with a blank crown attached to the rods to loosen the sedi-

ment, the pump forcing it with the water to the surface.

Then 150 ft. of old drill rods were dropped down the hole. The main rods were next lowered with the machine and it was found that the bottom of the old rods had stuck in the hole 10 ft. from the bottom. The main rods were then raised and 425 ft. of old rods were screwed together, on top of which 200 ft. of 1¼-in. piping were screwed and the whole length of 625 ft. dropped down the hole.

The reason for screwing this piping on the top of the old rods instead of using the whole 775 ft. of old rods was because the old drill rods were magnetic to a certain extent and would affect the needle of the survey instrument used, whereas it was found that the ordinary water piping had no appreciable magnetism.

To the top length of the 1¼-in. pipe a pilot wedge was attached. The position of this wedge was surveyed and the main deflecting wedge and guide wedge set so that the deflecting cut would be immediately opposite the direction in which the borehole was dipping.

Drilling was then re-started with a "C" bit and rods (1⅝ in. diameter), the deflection from the old hole commencing at a depth of 5872 ft., and was continued down to a depth of 6160 ft., where the footwall of the reef was again intersected.

From the above it will be seen what an enormous advantage is gained by means of this ingenious contrivance of a deflecting wedge, as in this case a second complete section of the reef was obtained at a distance of 60 ft. horizontally from the point where the reef was intersected in the original borehole, for the cost of drilling only 278 ft., as against 6160 ft. had a second hole been sunk from the surface.

The necessity of reducing the diameter for the deflected portion of the hole has now been overcome by an ingenious invention of William Gallagher, who for many years has been one of the leading diamond drill contractors on the Rand, where, I think I am right in saying, diamond drilling has attained a high standard of perfection. By Mr. Gallagher's invention, a succession of deflections can be taken from a borehole and each deflected portion kept the same diameter as the original hole. This was successfully carried out in the Witpoort borehole under my immediate supervision, where four successful deflections were made down to a depth of 1500 ft. owing to the hole either deviating from the vertical or taking a wrong direction.

Mr. Gallagher's invention is worked as follows: When the deflecting wedge has been set, the "C" hole is only drilled to a depth of about 12 ft. below the top of the deflecting wedge, after which the rods are withdrawn and the taper tap is then lowered and screwed into the guide ring at the top of the main deflecting wedge, which is pulled out of the hole by this means; after which, the pilot wedge, together with the bottom pipe connected to it, is also raised.

A piece of round iron about 1 in. in diameter, and in this case about 13 ft. long, the bottom end of which has been bent about ¼ in. out of the straight, is fastened on to the end of the main rods with two copper rivets, and lowered down the hole until the bottom end of the round iron is level with the top of the new hole which has been drilled by the "C" bit.

The rods are then lowered very gently and twisted round at the same time so as to get the bent end of the round iron into the new hole. Once the operator thinks this has been done, the rods are further lowered until the bottom end of the round iron rests on the bottom of the new hole. When this has been effected, the copper rivets are sheared off in the same way as explained above, and the main rods withdrawn, leaving the iron rod at the bottom of the new hole.

A fairly liquid mixture of portland cement is now made and put into a piece of piping of a length sufficient to carry the required amount of cement to fill up both the old and the new holes to about 1 ft. above the top of the iron rod. The bottom end of the pipe is burred on the inside so as to hold a blank flange, made of thin insertion, the same diameter as the inside of the pipe.

The pipe is then filled with a liquid cement and a similar insertion flange fixed on the top, but having a flap valve cut into it. The pipe is then fastened on to the end of the main rods and lowered down the hole until the bottom of it is about 1 ft. from the top of the iron rod. Water is then pumped down the center of the drill rods, the pressure opening the insertion flap valve downward and forcing the cement, together with the bottom insertion flange, out of the pipe, and thus filling both old and new holes with cement up to the requisite height.

This cement is allowed to harden for two or three days, after which, drilling with a "B" bit (the same size with which the original hole is drilled) is commenced, and the cement is drilled away until the top of the iron rod enters the hole in the bit and acts as a guide, pulling the bit and rods over from the old hole into the new. Drilling is continued to some depth below the bottom of the iron rod; when once the core breaks, the iron rod, together with the core, which has broken off underneath it, is drawn out of the hole in the core-barrel in the usual manner.

Subway Travel in London is not increasing at the rapid rate that it is in New York. The passengers carried on the London "tubes" in 1911 were 175,000,000, as compared with 173,000,000 in 1910.

The Genesis of the Steel Eating Worm

We noted in ENGINEERING NEWS of April 11, 1912, p. 711, the reappearance of that ancient Munchausen tale concerning "the worm that eats steel rails," which has for a quarter of a century been revived at intervals to do duty as a "filler" in stray corners of the newspapers.

As a result of that publication we have learned where this particular story originated. At our request, E. G. Spilsbury, Past-President of the American Institute of Mining Engineers, who was not only present at the birth of the story, but appears to have been a sort of associate midwife, has consented to reveal to the world the true record concerning this marvellous tale.

Mr. Spilsbury writes us as follows:

Sir—Your request for an account of the discovery of the "Steel worm" takes me back to the little manufacturing village of Haspe in Westphalia, in the year 1866. Just about that time the introduction of bessemer steel rails had begun on a commercial scale, and the press—both lay and technical—was full of discussions for and against the new product.

The professional employees of the different iron, steel and chemical works of the surrounding country used to meet every Thursday evening in the "Weinstube" of the local hotel and discuss all questions that were then of general interest. Any visitors were also made welcome. Under the stimulus of good fellowship, and possibly good wine, all sorts of theories—scientific and otherwise—were evolved, and the outcome of these meetings was the founding of the Society of "Ulk," which soon became known all over Germany as the source for practical jokes. The title "Ulk," an old and nearly obsolete German word, means "fun," and for our motto we used the three letters U L K as the initials for the words, Unsinn, Folly; Leichtsinn, Frivolity; Kneipsinn, Conviviality; and we never belied either one of them.

It was then at one of these Thursday evening meetings that the "steel worm" was born. An inquisitive editor from one of the neighboring towns was the subject of the joke, which was propounded in confidence to him as a scientific discovery of great value. It was supposed to be kept a secret until further investigation should prove the correctness of the theory.

By the following Thursday, the animal itself was produced, being a gruesome-looking thing, nearly all head, made out of gutta percha burette tubing. It was exhibited in a bottle half filled with roll scale, and the worm itself, having been filled with dilute hydrochloric acid would, when properly stirred with a glass rod, emit a drop, which at once attacked the flakes of the roll scale and proved the theory that corrosion of the rails was due to an acid secretion and that the resulting oxides were the food of the worm.

The newspaper man swallowed the whole story and proceeded to publish it, quoting the names of quite a number of well known iron and steel men as sponsors. The press all over Germany

took the matter up and very funny discussions resulted among their different correspondents. Some of the most amusing were the denials of the authorities quoted in the first published account that they had ever noted or studied this question of rapidity of corrosion of Bessemer steel rails, over those made from iron. Of course, the matter was too ridiculous to be kept up as a serious discussion for very long, but the story has never died, and every year or so crops up as a new and wonderful discovery.

While the carrying out of the details of the story was the result of the combined efforts of our society or club, the first conception of it was evolved from the brains of a young man whose name I forget, but who was, I think, a chemist at the Hoerde Iron Works in Bochum.

Quite a number of other discoveries and inventions emanated from that little coterie of bright intellects, but none of them have equalled the vitality or longevity of the "steel worm."

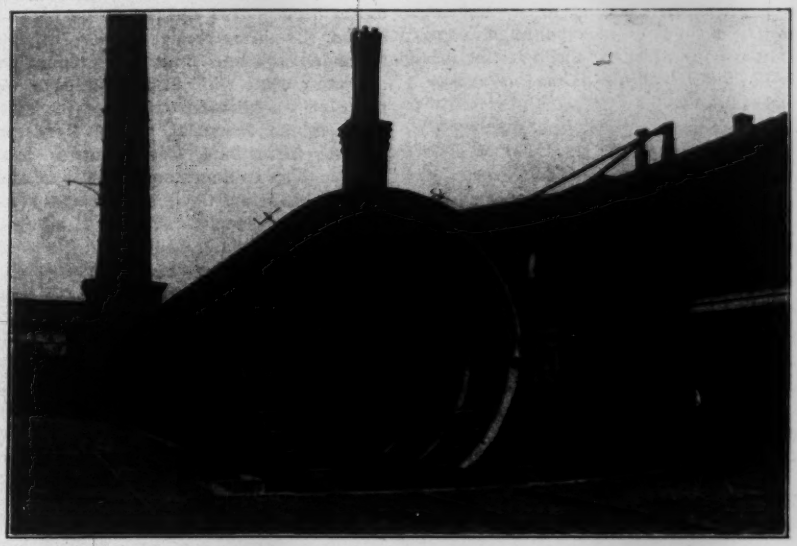
E. Gybbon Spilsbury.

45 Broadway, New York, Aug. 6, 1912.

An Apparatus for Disinfecting Railway Cars

The accompanying halftone is a view of a tank now in use on the Prussian State railways for the disinfection of railway cars. The tank is an air-tight wrought-iron cylinder, 16.4 ft. in diameter and 75.5 ft. long, firmly fixed on a foundation and provided with rails of standard gage so that it can take inside a railway coach of the ordinary European dimensions. It is provided, as shown, with a rolling door which can be hermetically closed against the open end.

The disinfection is carried on by the combined use of reduced pressure, elevated temperature and the application of a germicide in vapor form. The car is run into the tank and the door hermetically closed. The temperature of the interior is then raised to about 120° F. by means of steam pipes, which line the tank and at the same time pumps exhaust the air to about 27 in. of vacuum. At this temperature and pressure water boils and it is claimed that the combined action of the temperature and pressure causes the death of all vermin. To destroy all bacterial life, some germicide, such as formalin, is then introduced in vapor form from a small tank inside the main tank.



A Tank for Disinfecting Railway Cars Used on the Prussian State Railways

In operation it takes about five hours to effect complete disinfection of the car. One of these tanks has been in operation at Potsdam for some three years and is reported to have given complete satisfaction. The cost of the entire structural portion of the apparatus was $20,000, and the operating machinery, $1100. The cost of operating per day of 10 hr., not including interest and depreciation is given as follows:

Fuel (peat)	$2.00
Electricity (pumps)	1.00
Formation	1.10
Labor	0.90
	$5.00

The apparatus was made by the Julius Pintsch company at Hamburg. It is described in "Rivista Tecnica delle Ferrovie Italiane," July 15, 1912.

Letters to the Editor

Shipping Cement in Bulk

Sir—A brief account of the methods pursued in successfully handling a large quantity of cement in bulk may be of interest, in view of the letters on the subject which have recently appeared in Engineering News.

The H. E. Talbott Co., of Dayton, Ohio, in the construction of Lock No. 5, on the Monongahela River, at Brownsville, Penn., used over 60,000 bbl. of cement, practically all of which was shipped in bulk. The cement was received in ordinary box cars and wheeled into the cement warehouse through covered runways. The warehouse was in effect four wooden hopper bins with a light frame and sheet metal structure for protection from the weather. Tongued and grooved 2-in. lumber was used for the bins. Such crevices as appeared on account of settlement were covered with strips of tin and no appreciable leakage occurred.

The cement was taken from these bins in a motor-driven hopper car of about 15 bbl. capacity, which received its load through short spouts from the bottoms of the bins, an ordinary gate valve with rack and pinion being used. This hopper car delivered the cement in turn to a storage bin of 50 bbl. capacity at the mixer by means of an inclosed bucket elevator. The storage bin had discharge spouts with rack and pinion gate valves. The proper quantity of cement for each batch of concrete was received in a small metal tub mounted on wheels and pivoted to discharge into the hopper over the mixer.

The loss of cement due to this process of handling was negligible, certainly not as great as ordinarily occurs when bags are used. In addition the losses due to damaged and lost bags and the expense of returning the good ones are eliminated.

A change in the plan of prosecuting the work made it necessary to keep a considerable quantity of cement through the winter. On account of the prevailing dampness in the river valley during that period considerable anxiety was felt in view of the possibility of deterioration. However, the only effect noticed was the formation of a very thin film

over the exposed surface in the bins and an occasional small lump or cake where drops of rain had come through holes in the metal roof.

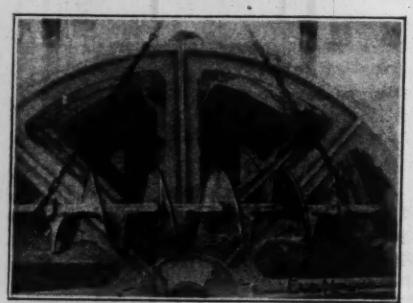
There is no doubt that the shipping of cement in bulk is entirely feasible, and where the quantity to be used will justify a small extra expense in respect to plant for handling the material there is no doubt that the method is economical as well.

C. M. STARKEY,
Chief Engineer, the H. E. Talbott Co.
Sault Ste. Marie, Ont.,
Aug. 19, 1912.

A Difficult Steel Casting

Sir—In your issue of Aug. 15, the writer noted, with great interest, an article entitled "A Difficult Steel Casting." We are enclosing a photograph of a carbon-steel casting recently completed in our works, trusting that you may consider the job to be of sufficient interest to make note of.

You will note that the angle of the helix is greater than that of the castings described in the article referred to, making the castings more difficult of accomplishment.



CAST-STEEL SCRAPER CASTING MADE BY THE WHEELING MOLD & FOUNDRY CO. WHEELING, W. VA.

The casting weighed 2100 lb. and its dimensions were as follows:

Length over all	10 ft.
Length of helix	7 ft. 10 in.
Diam. of helix	30 in.
Diam. shaft, center	4½ in.
Diam. shaft, ends	3½ in.
Thickness helix tip	¾ in.

This road scraping casting was made of very tough steel, and photograph shows it as it came from the sand.

F. DU P. THOMSON,
Chief Engineer, Wheeling Mold & Foundry Co., Wheeling, W. Va., Aug. 21, 1912.

Spiral Reinforcement a Necessary Safeguard for a Concrete Column

Sir—So many elaborate theories have been written and contradictory discussions advanced on the subject of concrete columns reinforced with longitudinal rods and circular or square hooping, it occurred to the writer that it might be of interest to the profession to relate an experience recently had during the erection of a four-story reinforced-concrete building.

After the roof of the building referred to had been poured and the falsework removed, the writer received a very urgent telephone message from the foreman at the job to come with all possible haste to make an examination of one of the columns in the basement, which, from all exterior indications, was about to collapse. The column mentioned extends 3 ft. 6 in. below the ground surface of the basement floor, on account of an elevator pit immediately adjoining.

After the fill had been entirely removed from around the column, it was found that part of the concrete, from the surface of the ground down to the top of the footing, had actually fallen away and in some cases the hooping left exposed. The concrete that remained outside of the hooping could be readily removed with the hand. The situation seemed indeed very alarming, and if the writer had not possessed a great deal of confidence in the efficiency of spiral hooping, an immediate exit would have been made to await the crash and subsequent removal of the débris.

By referring to the sectional elevation in the accompanying cut it will be noted that the cross-section of the column affected was 20x23½ in. The column is reinforced with six 1-in. round longitudinals, with ⅜-in. round spirals spaced 3 in. c. to c. and 16 in. in diameter. Originally the column was to have been 20x20 in., but on account of certain changes being made in the dimension of stairway, the column was extended in one direction to meet the architect's requirements.

The actual calculated dead load supported by the column, when its condition was discovered, amounted to 85,000 lb. The area of the 16-in. core is 201 sq.in., which gives a unit pressure of 422 lb. per sq.in. Notwithstanding the fact that the concrete could be crumbled in the hand, no settlement of the building could be detected. The concrete from *cd* up was in perfect condition and gave considerable difficulty to laborers while chiseling shoulders to receive shores, necessary before a sufficient number of men could be induced to make the repairs.

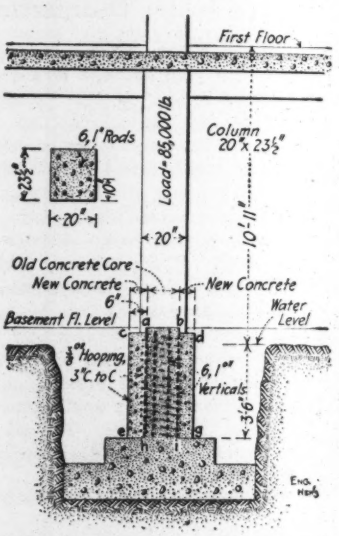
The column was repaired by removing the concrete outside of the hooping and placing 6 in. of new concrete from *cd* to *eg*, reinforced by additional hooping as shown in sketch. Particular care was exercised in obtaining a perfect shoulder in the old concrete in plane *ab*.

A thorough investigation was made to ascertain the cause of disintegration. At first it was thought that the seepage water surrounding the columns was accountable. Samples of the water taken by R. W. Hunt & Co. were tested and gave the following results:

Total solids	114.89 grains per U. S. gal.
Organic	21.70 grains per U. S. gal.
Alkalinity	0.2864 grams of sodium hydrate per 1000 c.c. water
Nitrogen as nitrite	0.014 gr. per 1000 c.c.
Nitrogen as nitrates	0.100 gr. per 1000 c.c.

Neither the alkalinity, the organic matter, the nitrates or nitrites seemed sufficiently high to account for any decomposition of the concrete during the period after pouring. After making further investigation it was concluded that prior to the pouring of concrete into the column form, the seepage water was permitted to enter the excavated opening around the column, through negligence on the part of the foreman, causing a separation of the cement from the aggregate during the deposition of the concrete from the first-floor level.

If it is a fair question, the writer would like to inquire of the advocates of



CONCRETE-COLUMN BASE REPAIRED WITH NEW CIRCUMFERENTIAL CONCRETE

the non-reinforced concrete column if this column would have stood under the circumstances described, if it had been reinforced only with cement?

W. J. KNIGHT.
Wainwright Bldg., St. Louis, Mo.
Aug. 13, 1912.

Notes and Queries

In the letter on "The Autoclave Boiling Test for Cement," by Bernard Enright, in Engineering News, Aug. 8, 1912, the last cement-sand mixture on p. 264 should have been "1 cement to 6 sand" instead of "1 cement to 5 sand." Since the publication of the letter the tests reported in that table have been carried further, as follows:

TENSILE STRENGTH AT 28 DAYS

Neat	960 lb.
1 cement to 3 sand	505 lb.
1 cement to 4 sand	405 lb.
1 cement to 5 sand	310 lb.
1 cement to 6 sand	275 lb.

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Engineering News

The Seattle Municipal Electricity Supply System

The Seattle municipal electricity supply system after seven years growth seems to have earned an enviable place among municipal undertakings and indeed among dividend-paying public utilities whether under municipal or private management. The first year's operation, 1905, yielded a deficit of 2% on the indebtedness, $876,000. At the end of the fourth year, the 1.6% accumulated deficits had been wiped out, depreciation revenues, equal to 6% the value of the enlarged property, had been established, and a 1% surplus on the total indebtedness earned. At the close of the seventh year, 1911, the depreciation revenues had mounted to 21% the value of a still larger property, the annual surplus was 4.7% of the indebtedness, and the accumulated surplus 11%. During this time the cost of electricity had dropped from 20c. to 6 for ordinary consumers. The citizens may well have confidence in a department having such a record.

Several things contributed to this end—factors like a low rate of bond interest secured by pledging city credit, the readiness of the people to sanction needed enlargements, the re-investment of depreciation reserves and surplus earnings in lieu of bonds to that extent, and elimination of the evil effects of political interference. The 125-page report of the lighting department gives in greater detail all the facts we have here commented on or shown in our review elsewhere in this issue. This attractive and interesting document should be in the hands of all those engineers who believe in municipal ownership of utilities—and of those who do not believe in it.

An Expensive Method of Hydrographic Surveying

An uncharted shoal between Point Judith and Block Island, at the eastern entrance to Long Island Sound, was the cause of serious damage to the United States battleship "Nebraska," on Aug. 8.

It is not yet known just what the damage to the "Nebraska" will amount to in dollars; but it is safe to say that it will be no insignificant sum. The vessel was proceeding very slowly, otherwise its injuries might have been more serious, perhaps even the total loss of the ship. At all events this experience illustrates an expensive, but not uncommon method of "finding" uncharted or undiscovered shoals and other dangerous obstructions to navigation.

No part of our coasts has been more thoroughly surveyed by the time honored method of sounding with lead and line than this section about the Newport Naval Station. In general the charts of no foreign nation are held in higher repute among navigators than those issued by the U. S. Coast and Geodetic Survey. The underlying reason for this and other similar accidents comes from placing reliance on hydrographic surveys of comparatively shallow waters where depths have been verified by soundings only.

To find submerged boulders on a sandy bottom with sounding lines is good fortune rather than good surveying. It has taken many years and cost enormously in money and lives to find obstructions by running vessels upon them. One of the best reasons why shipwrecks from this cause are not more frequent is because deep-draft ships seldom leave regular paths or channels which have been proved by the passage of many ships to be free from obstructions.

There is another and more economical method of verifying charted depths than maneuvering $10,000,000-battleships over them. This is by sweeping or dragging over them with the long wire drag developed and already used extensively by the Coast & Geodetic Survey, a description of which will be found in ENGINEERING NEWS of Dec. 1, 1910. A party under Assistant N. H. Heck is even now engaged in a wire-drag survey in the immediate vicinity of the place where the "Nebraska" struck, and in the course of another season's work would undoubtedly have found that particular boulder.

A charted shoal does not insure against shipwreck, as the recent grounding of the battleship "Arkansas," near Rockland, Maine, illustrates. The accident occurred on June 2, last, during the speed trials of the battleship and before its final acceptance by the government. While in charge of a coast pilot employed by the ship's builders, the "Arkansas" scraped its bottom on a ledge now known to have been one of those found by a wire-drag survey in the season of 1908, and shown on all charts of the Coast & Geodetic Survey issued since. The pilot was using an old chart and hence did not know the existence of a dangerous ledge, not far from the trial course. However, such an accident could not have happened had the vessel been in charge of naval officers, who are most careful navigators, and among whom it is held to be hardly less than criminal to run a vessel upon a charted shoal.

The Question of International Specifications

Next week's congress of the International Association for Testing Materials in New York, its sixth general assemblage since 1897, and the first in America, is somewhat different from the majority of international congresses, both in aim and in method. The general object of exchange of opinion between the technical men of different nations is in this case supplemented by the larger and more far-reaching object of deriving active positive conclusions from the exchange of opinion. The association since its first meeting has made endeavors to unify not only the methods of testing, but also the standards set up for materials in the form of specifications. Consistent work has been done on this problem. In its methods also the association differs from other international-congress bodies, for it works not merely through its own centralized organization but through and with the assistance of national affiliated organizations in the principal countries. The American Society for Testing Materials and the German association of similar nature are among the largest of these.

The congress promises to be a remarkably active and successful gathering. Information obtained from the officials who have been at work on the preparations for months past leads us to believe that the quality of the work, the mechanical provision made for its due handling, the attendance from abroad and from the United States, and the interest brought to bear upon the important technical matters coming before the congress, will be such as to set the association meeting upon a remarkably high plane. Some 300 foreign members will be present, together with probably 500 or more Americans. The large list of papers to be presented contains essential contributions both to the knowledge of materials and to the art of testing them for various properties, and has intimate bearing on the industrial economy of each country, apart from any international aspect.

However, the most important element, from one point of view, is the prospective discussion of international specifications. The committee on internal specifications for iron and steel will present a full report, we are informed. This report, when it appears, will for the first time give us an intimation of the ultimate prospects of international unification of material quality. The indications on this matter during the past year have been discouraging. The international deliberations which have taken place, as the culmination of fifteen years' constant effort, have seemed to result in a frank admission of failure. The actual report of the committee at the congress and the deliberations thereon will tell whether this acknowledgment of failure is maintained, and whether, if maintained, it speaks for a real and permanent impossibility of coming to agreement on the principal qualities of the most largely used materials.

Viewing the matter from the outside, it is hard to understand why (for example) structural steel in Germany, in the United States and in England should mean three distinct kinds of material. There seems to be no reason why the materials should not be substantially identical, and why any minute differences (which may continue to exist, on account of industrial conditions in the several countries, varying ore and fuel supply, etc.) cannot be allowed for in minor clauses or covered by suitable explanatory citation of limits.

Relieving Sidewalk Traffic Congestion in New York City

The relief of street-traffic congestion is a problem which the municipal engineers of our large cities are coming to face more and more every day. In most cities it is the crowded condition of roadway traffic, the obstructions of interlacing street-car systems and innumerable vehicles, that must be regulated and reduced, but in portions of our older and more closely populated cities, particularly in the business districts, the problem is a sidewalk problem. In the business districts of New York City, that is, on certain streets south of 42nd St., Manhattan, the congestion of sidewalk traffic has become so great as to demand immediate remedy. In these streets, particularly in the extreme southern end of the island, the multiplied population of floor upon floor of tall skyscrapers throws onto the sidewalks at certain hours of the day many more people than their limited area can accommodate. The obvious remedy of widening the sidewalks at the expense of the roadway cannot be applied in these congested streets, for where the roadway is wide enough to stand this there is always a street-car track which would prevent the narrowing of the roadway limits and where there are no car tracks —on such streets, for instance, as Pine, Wall and Nassau—the roadway itself is hardly wide enough for the required vehicular traffic which passes through these streets in the early and late hours of the day and the whole space between the building line will not more than accommodate the human travel at the noon hour.

Since the widening of the sidewalks could not be made toward the roadway, the only remaining direction had to be followed. In the last two years the City of New York, acting through its Board of Estimate, which was given control of the streets in 1905, has approached a successful solution of the sidewalk problem by removing in a number of streets all the many obstructions which filled the sidewalk and prevented the free passage of pedestrians. In New York City, as in many other cities, the building line has been regarded as a surveyor's line to be generally kept to in a street structure, but of an extremely flexible nature as regards minor protuberances which do not occupy the entire width of the building lot. Bay windows, stoops, steps, entrances, show windows, stands and signs all overlapped the legal building line and encroached on the valuable space required for the sidewalk. The Board of Estimate some time ago decided that such encroachments were illegal and ordered their removal on a number of specified congested streets at the expense of the individual owning the encroaching property. At first the order was fought by some, but court rulings were frequently made that although permits to build encroachments may have been issued by one or another city department, the city's title to its streets is inalienable and it may repossess its street space thus given away.

Acting under these rulings, during the past year a large amount of sidewalk obstruction-removal work has been done. Borough President McAneny of the Borough of Manhattan has recently reported that in that borough during the past year 21½ miles of street widening has been completed and approximately 425,000 sq.ft. has been reclaimed by the city. The assessed values on these streets vary from $1000 to $10,000 a front foot, so it will be seen that the city has recovered valuable property. The expense to the city has been merely the widening of the roadway where the ground has been taken for that purpose, whereas the cost to owners is estimated to be between $4,000,000 and $5,000,000. In nearly every instance the owners have taken the action of the city in good part and have endeavored to readjust their entrances and fronts in a more fitting architectural style than they had before. The benefits have been most marked. On Fifth Ave. in particular and on lower Broadway, while there is still a very marked congestion in traffic, the situation is much improved over what it was before when the whole body of pedestrians had to turn out every 30 or 40 ft. to avoid some illegal obstruction to the sidewalk.

Other cities will do well to profit by the experience of New York and put a stop to sidewalk encroachments before they have gone any further. By this means large expense to private owners of property will be saved and the comfort, convenience, time and patience of the sidewalk-using public will be conserved.

The Invention of the Automatic Car Coupler*

In our obituary column a few weeks ago, there was recorded the death on June 16, at Alexandria, Va., of Major Eli H. Janney, the original inventor of the automatic car coupler now in universal use on American railways. It seems appropriate at this time to put on record some of the interesting history in connection with the development of the automatic car coupler. While the facts related were familiar to every railway man a quarter of a century ago, they are unknown to many of the younger generation.

Perhaps no mechanical problem has ever attracted the attention of a wider circle of inventors and engineers than did the problem of designing an automatic car coupler during the quarter century following the Civil War. At that time the link-and-pin coupler was in universal use on freight cars, although on passenger rolling stock the Miller square-hook coupler had generally superseded it. The foremost reason for the adoption of an automatic coupler was, of course, the matter of safety to employees. With the old link-and-pin coupler it was necessary for a switchman to hold the link with his hand and guide it into the mouth of the drawhead on the meeting car, as two cars came together. He had to be quick enough to withdraw his hand before the drawheads met or his hand or fingers would be caught and crushed. There are thousands of men still living who sacrificed fingers, hands and arms in coupling cars with the old-time link-and-pin.

It was easy to perceive the possibility of inventing some device which would allow two cars to couple automatically when they came together and which could be released by some mechanism that could be operated by the switchman from the side of the car. Inventors by the thousand and ten thousand, amateur and professional, in all parts of the country, labored eagerly to solve the problem. The amount of money spent on drawings, models, attorneys and patent-office fees and in attempts to manufacture and sell various types of automatic car couplings in the twenty years from 1865 to 1885 would probably reach many millions of dollars.

The greatest difficulty was that the invention was so easy to make! Almost any mechanic with a little ingenuity could contrive some arrangement which would work beautifully on a small model and would meet what was supposed to be the main requirements; to wit, automatic coupling by impact and release from the side of the car.

Thousands and ten thousands of amateur inventors, after scheming out a device to accomplish these ends, believed that they had solved the problem which had baffled everyone else and that untold fortunes were already within their grasp. The next step was to seek out the blacksmith, the country carpenter or the model maker and have some sort of model of the wonderful invention hammered or whittled out. Then a patent solicitor was hunted up and after being sworn to absolute secrecy, the model was unfolded to his gaze.

Thousands of patents in the files at Washington record the buried hopes of whole regiments of car-coupler inventors. Every one of them believed, and probably if living still believes, that his final failure to obtain the fortune which he deemed was rightly his was due to the brutality and ignorance of railway officials, their failure to appreciate the merits of his device and their deliberate preference to sacrifice the lives and limbs of their employees rather than spend money to provide safer methods of operation.

There was undoubtedly some measure of truth in this indictment. The railway companies and railway officers of that day did procrastinate in the adoption of an automatic coupler; and without doubt many of them failed to appreciate the vast importance of such a device, not only from the point of view of safety to their operatives, but from the point of view of economic railroading.

But there were numerous reasons for this procrastination of the railways. It was extremely difficult at that day to find anyone wise enough to say with authority what type of automatic coupler should be adopted; and even if one were found wise enough to make a choice, with the loose organization existing among railway companies at that day, there was great difficulty in making any such decision when once arrived at authoritative. It was for a long time a case of, "who shall decide where doctors disagree?"

There were, for example, a considerable number of mechanical officers who believed that any automatic coupler in order to be successful must be of the link-and-pin type in order that it might operate automatically with the cars already in service. In the late '70's and early '80's, furthermore, the matter of selection became complicated by the entrance of large business interests. Some of the automatic-coupler inventors secured capital and influential backing and carried their devices to such a stage of mechanical excellence that the officers of certain railways became interested in them and ordered the equipment of a considerable number of cars. In this way certain railways became committed to a coupler of one type and make, while other railways were equally interested in a coupler of different make.

Action in the Master Car Builders' Association toward the adoption or approval of any one type of coupler was blocked by the united opposition of those who were interested in other types. The prize at stake was believed to represent many millions of dollars. Year after year, committees of the Master Car Builders' Association investigated the merits and demerits of various types of couplers and conducted exhaustive tests to determine their mechanical excellence or defects; but when it came to the point of action by the Association to adopt any one type of coupler, the clash of opposing interests was sufficient to create a deadlock and prevent action.

It is of great interest to note that while this conflict and delay continued, a coupler which was later to become the standard for railway use had been invented and to a considerable extent perfected ready for adoption. Major Eli H. Janney, a former Confederate officer on the staff of Robert E. Lee, invented in 1873 an automatic coupler having a forked head and a hook or knuckle revolving in a horizontal plane. It was a crude device as shown in the illustration, Fig. 1, but it contained the germ which was afterward to be developed into a practical coupler. In the following year, 1874, Major Janney patented an improved coupler embodying the same operative elements but worked out on much better lines. Reference to the accompanying cut, Fig. 2, will show the close resemblance to the present type of coupler.

Three years later, in 1877, Janney succeeded in interesting in his invention the McConway & Torley Co., of Pittsburgh, who were owners of a malleable-iron foundry, and they coöperated with him in developing and marketing the invention. No couplers of the design shown by Janney's patent of 1873 or 1874 were ever put in service; but in 1879 he took out a new patent in which the contour lines of the meeting surfaces were changed to those that were eventually adopted by the Master Car Builders' Association, and some couplers of this design were soon tried on passenger cars. The locking mechanism of the 1879 coupler, however, was not adapted for use on freight cars, and three years later, in 1882, Janney took out an additional patent for a coupler having the same contour lines, but a vertically moving locking pin especially suited for freight-service conditions. The same year witnessed the first trial of the invention on freight cars, the McConway & Torley Co. equipping 50 cars with the device, which were put in service, we believe, on the Pennsylvania R.R.

It is worth noting that a large amount of courage was required for the initial adoption of this new and hitherto untried invention. In the first place the railways

*For the illustrations reproduced to accompany this article, and some of the facts stated, we are indebted to a pamphlet entitled, "The Evolution of Car Couplings," issued by the McConway & Torley Co., of Pittsburgh, in connection with the International Railway Congress of 1905.

using it had to look forward many years in the future to receive any benefit from its installation. It must be remembered that with this type of coupler, automatic coupling could only take place when both of the cars which met were equipped with the coupler. But with cars running in general freight service, a few hundred or even many thousand cars equipped with automatic couplers were lost among the freight-car rolling stock of the country, which even at that date numbered nearly 1,000,000 cars. Moreover, the risk to life and limb in coupling the new drawbar with the old link-and-pin coupler was even greater than in coupling two link-and-pin cars together. This increased risk had to continue for all the years to come until (if that time ever came) a large percentage of the rolling stock of the country should be equipped with this coupler.

Meanwhile other rival couplers were securing a strong position and influential backing. Those favoring the automatic

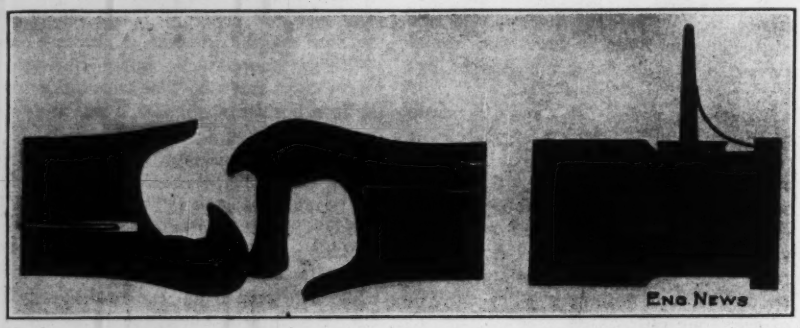

FIG. 1. AUTOMATIC CAR COUPLER PATENTED BY MAJOR E. H. JANNEY IN 1873

link-and-pin coupler as the standard had the very strong point in their favor that their device would work with the old hand-operated link and pin, if not automatically, at least with less danger to employees than would a coupler of entirely different type.

Probably the most serious indictment brought against the Janney type of coupler, however, and the one which had the largest weight with the majority of railway officers of that day was the argument that the "loose slack" of the link-and-pin coupler was a necessity in starting trains. It was argued with every show of reason that since the maximum resistance in the movement of a railway car is at the moment of starting, it was necessary that the locomotive should be able to start one car after another, as actually happened in the starting of a freight train coupled with the old link and pin. Hence, if a so called "close coupling" were adopted, where the locomotive had to start the whole train at once, a serious limitation of the load a locomotive could haul would be at once established.

This argument was absolutely convincing to the majority of the railway operatives of that day, and indeed it is probable to a majority of the railway managers and superintendents as well, coming up, as most of them had at that time, from the ranks.

It was fortunate indeed for the railways of that day and not less fortunate for the general public, whose interest was as great as that of the railway companies themselves, that there were among the mechanical officers of railways at that time, certain men of large ability, with a broad grasp of mechanical principles and with sufficient public spirit to be willing to devote time, energy and influence to the correct solution of a problem whose importance they fully realized. Among the names which stand out with greatest prominence among the leaders of that day who had the largest influence in bringing about a correct decision are those of the late M. N. Forney, for many years Editor of the *Railroad Gazette;* Godfrey W. Rhodes, Superintendent of Motive Power of the Chicago, Burlington & Quincy R.R., and E. B. Wall, Superintendent of Motive Power of the Pittsburgh, Cincinnati, Chicago & St. Louis Ry. Mr. Wall was among the early graduates of Stevens Institute at a day when graduate mechanical engineers in the railway service were little appreciated. But he made his abilities so felt that his promotion was phenomenally rapid. Within a few years of his graduation, he was placed in full charge of the motive power and rolling stock of the Pennsylvania's Southwest system.

It was Mr. Wall who succeeded in finally ending the long deadlock in the Master Car Builders' Association by a motion in the annual convention of 1887 that the vertical-plane type of coupler should be adopted as the Standard of the Association.*

To understand the significance and importance of this term, it is necessary to recall one of the chief objections to the old-time link-and-pin coupler, which was the fact that in order to couple a loaded car to an empty car it was often necessary to employ a bent link on account of the difference in height of the two cars with the journal-box springs compressed different amounts. By vertical-plane type of coupler, therefore, Mr. Wall meant a coupler which would permit the two hooks or parts engaging with each other to move up and down vertically without introducing stresses in the couplers. The Janney type of coupler might more properly, perhaps, have been called a "horizontal-plane" coupler since its moving parts revolve in a horizontal plane to engage or release; but Mr. Wall's motion not only prevailed in the meeting where it was proposed, but it gave to the coupler a distinctive name.

*In the pamphlet issued by the McConway & Torley Co., referred to above, Wm. McConway, to whose business foresight and energy a large measure of the success of the Janney coupler was due, gave the following reminiscences concerning the origin of the term "vertical phone":

"Questions are frequently asked as to the origin of the phrases "Vertical Plane" and "Master Car Builders Coupler."

The first of these originated with John W. Cloud, Engineer of Tests at the Altoona Shops during the discussions of the coupler question incident to the effort being made to select a type of coupler which could be made standard, to replace the "Link and Pin."

At the time, there were under consideration various link controlling devices; combinations of links and hooks; "harpoon," or arrow head type; solid draw hooks which coupled torsionally; with the "Miller" and the "Janney" as the two used on passenger equipment to the exclusion of all other designs.

In a discussion of the subject, Mr. Cloud made the remark that the only automatic couplers which, so far, had given evidence of practicability by successful service were those in which the contact faces of the couplers came together on a "vertical plane." The phrase "took," and hence the "Miller" and "Janney" became known as "vertical plane couplers."

To Godfrey W. Rhodes belongs the paternity of "M. C. B." as applied to couplers. When after several years of experimental service, it had been demonstrated that the "Janney" met the requirements laid down for a freight car coupler, as did also some made in imitation (and it had been truthfully said "the nearer the imitation, the better the coupler") there came a difficulty in the way of an official declaration of approval by the Association. In that the Janney patents controlled all couplers that would couple with it. To remove this difficulty, the owners of the Janney patents, at the request of the M. C. B. Association, waived their rights to such members of the Association as applied for the waiver to permit the use of the Janney lines for couplers for other makes, for freight cars only. All the members of the Association applied for and received, individually, deeds of waiver. This, however, did not remove all the difficulties in the way. To recommend the adoption of the "Vertical Plane" type would not cover the case, for the "Miller" coupler was of that type, and was not available. There was no other name sufficiently descriptive except "Janney" and that in the minds of some, at that time, seemed too much credit to give to the originator of the type, and would tend also to the overshadowing in the market of the imitations. At this juncture, Mr. Rhodes, at a meeting of a railway club in the West, made certain remarks which at this distance of time the writer can only paraphase: "If," said he, "you are unwilling to designate the type which has been selected as the 'Janney' type, then call it by any other name which will make the identification effective. Call it the 'Master Car Builders Coupler' if you will, but call it anything so that the name will identify the thing which has been adopted."

But the one man who stood in the forefront with those of "the Faith," and whose belief never faltered, whose clear judgment always saw the time for action through the clouds of opposition, and whose recommendations never proved a step backward when put in effect, who never hesitated to submit the soundness of his own conclusions to the test of the knowledge and experience of others before his final recommendations, was Edward B. Wall. Others there were and many of them too, but his name heads the list, close seconds though they be."

There were plenty of able engineers in the mechanical department of the railways of that day, however, who realized that the vertical-plane feature of the Janney coupler was not less important than the fact that the coupler substituted "spring slack" for "loose slack." However advantageous the "loose slack" was in enabling the light locomotives of the '70's to start a long train of light cars, it had already been found that with the increasing weight of rolling stock and increasing train lengths, the shocks from this loose slack in train movement were rapidly becoming ruinous both to cars and to lading. Breakage of links, pins and bull-nose drawbars on the roads of heavy traffic in the early '80's had become so serious an expense to the railways, to say nothing of the wrecks caused by break-in-twos, that many progressive railway officials were ready to become converted to the new doctrine of close coupling and spring slack in place of loose slack.

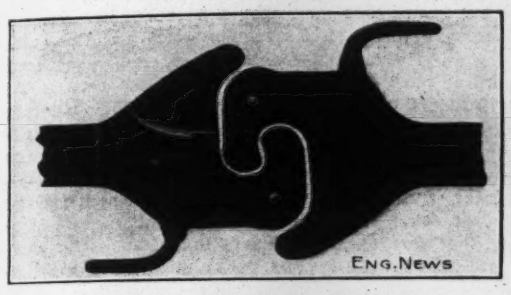

FIG. 2. COUPLER PATENTED BY JANNEY IN 1874

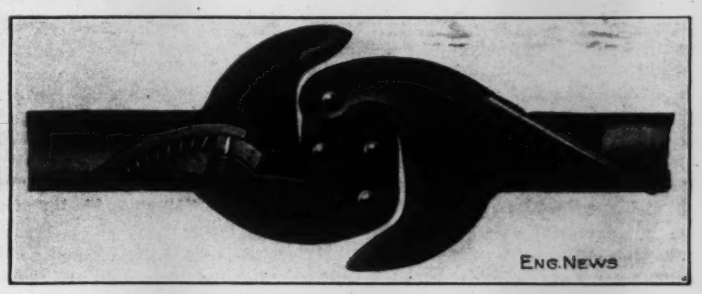

FIG. 3. COUPLER PATENTED BY JANNEY IN 1879
(Contour lines of coupler afterward adopted as standard by Master Car Builders' Association)

Thus it came about that while there was strong opposition to the movement to make the "vertical plane" type of coupler the standard it was not sufficient to defeat the measure. There was, however, a very strong opposition to making any one patentee's or manufacturer's coupler the standard for the railways of the country. This difficulty was finally met and overcome by the Master Car Builders' Association Committee, which made a contract with the owners of the Janney patent by which they relinquished their claims to a monopoly as to the contour lines of the Janney coupler and left them open to the free use by any manufacturer or railway.*

With the difficulties thus cleared away, it would seem that action by the railways should have been prompt and rapid to equip their cars with the new coupler. Procrastination, however, continued. A large number of important and influential railways which had opposed the new coupler continued their opposition and refused to equip any of their cars with it. This attitude was distinctly discouraging to the railways which had supported the new type of coupler. While progress was made year by year in the equipment of cars, the work proceeded at a discouragingly slow pace. It became evident that unless some influence was brought to bear upon the dilatory and delinquent railway companies, the process of changing from the old coupler to the new would drag along for many years. During all that period the benefits which should be obtained from the millions of dollars already invested in the new equipment would be largely lost. Still more important from the point of view of railway economy, incomplete equipment with the new coupler delayed the increase in weight of cars and weights of trains which most progressive railway officers had by this time perceived to be the main road to economic railway transportation.

It should be remembered that at this time the power of one railway to influence the action of another was extremely limited. The only general organization of the railway companies was the rather loose one known as the General Time Convention, the predecessor of the present American Railway Association. This organization adopted the Master Car Builders' type of coupler as standard in 1890, but it had no more power of compulsion over its members than had the Master Car Builders' Association. Tables published in ENGINEERING NEWS of Dec. 21, 1889, showed that at that time only a little over 60,000 cars in all had been equipped with the Master Car Builders' type of coupler, and there were then in service nearly 18,000 cars equipped with automatic couplers of other types. Still more important, a number of railways reported that they were even at that date equipping all their new freight cars with these rival types of automatic couplers.

It was under these conditions that one man, imbued with the spirit of public service, became convinced that the only prompt solution of the deadlock into which the railways had fallen was national legislation to compel the railway companies to equip their cars with automatic couplers.

This man was L. S. Coffin, of Iowa. Mr. Coffin was a clergyman when, in the early '80's he was appointed by the Governor of Iowa to membership on the State Railroad Commission. It was while in this

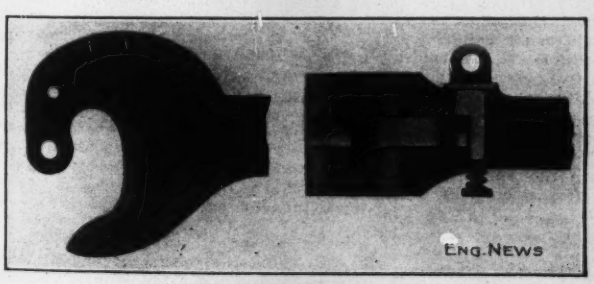

FIG. 4. COUPLER PATENTED BY JANNEY IN 1882

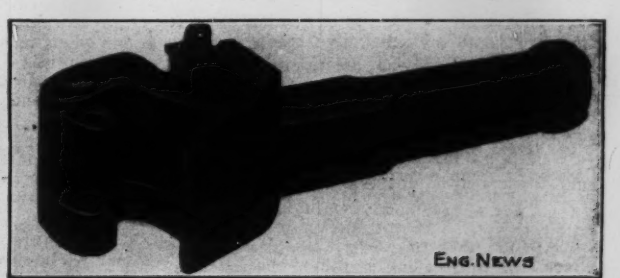

FIG. 5. FINAL FORM OF JANNEY COUPLER

*The opening of the field to competition caused a host of coupler manufacturers to enter the business and offer M. C. B. couplers. The fact that the owners of the Janney patent were able nevertheless, to retain the lion's share of the business, is shown by the statement in the pamphlet referred to that up to 1905 they had made more than 1,400,000 Janney couplers, besides 100,000 of a modified Janney type and large numbers of still other couplers with other modifications.

position that he became familiar with the slaughter of railway employees in the operation of the link-and-pin coupler and the hand brake. After the expiration of his term of service as a railroad commissioner, he undertook a crusade to bring about a national law for safe railway appliances. While he had the support and approval of the railway labor unions, he worked practically alone. He had already made himself thoroughly familiar with the brake and coupler situation and had taken part in the famous brake and coupler tests in Burlington, Iowa, in 1887. For years he spent his time traveling all over the country, interesting men of influence everywhere in his campaign for the safety of "his boys," as he called the men in the railway train service. His honesty, his zeal and his thorough knowledge of the subject secured him a hearing everywhere and disarmed opposition. It was, without doubt, due to the work done by L. S. Coffin more than to any other one man that Congress enacted, in 1893, the first railway safety-appliance act, which gave the railways five years to equip their cars with automatic couplers and with sufficient power brakes so that trains could be operated by brakes controlled on the locomotive without the use of hand brakes. It also required a standard height for drawbars, to be fixed and adhered to by the railway companies, which, it can be readily seen, was a necessary feature for the safe installation of the M. C. B. type of coupler.

While Mr. Coffin's object in his crusade for the national safety-appliance act was a humanitarian one, the act merely compelled the railways to do with reasonable promptness what it was enormously for their own interest to do. He is fully entitled to the distinction of having been instrumental in making possible the saving of more millions of dollars in railway operating expenses than can be put down to the credit of many officials of great railway systems.

It seems to us that this record of the introduction of the Master Car Builders' coupler into railway practice is not only full of interest as a matter of engineering history, but contains many lessons valuable to the engineer and the railway officer of the present day.

It is clear now, for one thing, that the railways lost many millions of dollars and sacrificed the lives and limbs of many thousands of their employees by delaying for 15 to 20 years the adoption of the automatic coupler after it had been invented and brought to a practical stage. There were mechanical engineers in the service of the railways in the late '70's who could have taken Janney's device of 1874 and have developed a thoroughly practical automatic coupler suitable for general use on freight cars.

The two great difficulties which stood in the way of progress at that time were the lack of any organization among the railways sufficiently powerful to enable joint action to be taken on matters of mutual interest and the prevalence of the graft system among too many of the old-time railway officers, under which the adoption of any mechanical device was only possible when the officers interested were taken care of.

It is, on the other hand, a great tribute to the sound mechanical judgment of the influential men who brought about the adoption of the Janney type of coupler that they should have had the wisdom to select a coupler which would be sufficient not only for the rolling stock of that day, but should be capable of development for the enormous train loads and stresses required by the railway practice of a quarter-century in the future.

In illustration of this point it may be remarked that the present standard Master Car Builders' specifications for automatic couplers requires them to withstand without injury a pulling test of 150,000 lb. Undoubtedly had Janney or any other automatic-coupler inventor of the early '70's been asked to submit their device to any such test, he would have considered the feat a mechanical impossibility.

It is, of course, true that the present perfection of the coupler has been due to 25 years of evolution in design, in materials and in details of application. However, had a type of coupler been chosen through mistaken mechanical judgment 25 years ago, which was incapable of such evolutionary development, the loss to the railways of the United States might have been measured by untold millions.

State Supervision of Water Power Development in Pennsylvania

In connection with the proposed hydroelectric development on the Clarion and Allegheny rivers in Pennsylvania, the largest scheme ever projected in that state, the State Water Supply Commission has made an exhaustive study and investigation to devise means of safeguarding the interests of the public. The provisions specified by the Commission for this purpose are summarized in the July issue of the *Journal* of the Engineers' Society of Pennsylvania, as follows:

(1) That the company shall not construct, erect, or build any dam, wall, wing-wall, wharf, pier, embankment, abutment, projection or other obstruction, in or along any river or stream, nor in any manner change or diminish the course, current or cross-section of any river or stream unless and until the company shall have submitted to the Water Supply Commission of Pennsylvania complete maps, plans, profiles and specifications and such other information and data as the Commission may deem necessary relating to any proposed dam, wall, wing-wall, wharf, pier, embankment, abutment, projection or other obstruction, and a majority of the members of the Commission shall have approved the same.

(2) That within 12 months after the date of the issuance of Letters Patent there shall be presented to the Water Supply Commission of Pennsylvania the following data, determined by actual physical inspection, test and survey:

(a) An accurate map of the area flooded by each dam, with contours sufficient to calculate the quantity of water impounded at various depths, also revised data concerning rainfall, runoff, available storage, and the effect upon them by drought or freshet conditions. Also the extent to which the reservoirs formed can be used to ameliorate floods in the Clarion and Allegheny rivers.

(b) Design of each dam with spillways, drains, and other appurtenances, showing its proportions, with stress diagrams, demonstrating the stability and adaptability to the site selected.

(c) Definite information obtained by suitable investigation concerning the character of foundation upon which the dam will rest, and of the geological structure of the side slopes of the valley.

(d) Method proposed for utilizing the water power, in sufficient detail to advise the Commission of the location and character of power station, with its equipment, and of the distribution of the power developed.

The data as to foundation, side slopes, proportion of dam, spillway, power station, and other appurtenances, to be accompanied by reports from at least two engineers of recognized standing, and familiar with similar structures as to their stability, sufficiency, and adaptability for the purpose and location indicated.

(3) That the requirements of the Corps of Engineers, United States Army, in charge of the Allegheny river, as to the minimum stream discharge, must be embodied in any plan for using the water stored, as well as the rights of lower riparian owners to have available at all times at least the minimum stream flow, as determined by the Commission, must be protected.

(4) That the construction of the dam must be under the personal supervision of an engineer skilled in this branch of his profession, who must be constantly represented by competent inspectors to insure that the construction is carried out in strict conformity with the plans and specifications which may be approved by the Commission; this to be entirely independent of such inspection and oversight as the Commission elects to give to the work.

(5) That the company hereinabove named must satisfy the Commission, by written testimony, before the expiration of 18 months from the date of the issuance of Letters Patent, of their ability to carry the construction forward promptly and continuously, a requirement which is demanded by the influence which the proposed improvement will have upon the property values affected.

(6) That the operation of reservoirs, in so far as the control of floods and the maintenance of low water flow is concerned, shall be subject to the direction and jurisdiction of the Commission.

(7) That in disposing of the power produced by the said company, the preference shall be given to individuals, corporations or communities within the State of Pennsylvania upon equal terms and conditions of payment for the same.

(8) That this charter shall not go into effect until the company shall accept the same, by its proper officials, covenanting at the same time that a failure to comply with the conditions hereinabove named shall work a revocation of its privileges by said charter granted; said aceptance shall be filed in the office of the Commission, and a copy thereof in the office of the Secretary of the Commonwealth, within 30 days after the issuance of Letters Patent by the Governor of the Commonwealth.

The Wisconsin River Flood: Its Causes, Lessons and Repair Work

By D. S. Burnett*

A month has passed since the upper Wisconsin River valley was visited by the greatest flood of its history, which was disastrous to nearly all the manufacturing interests along the Wisconsin River and many plants on the tributary streams between Tomahawk and Mosinee, a distance of about 50 miles north and south. No lives were reported lost; but such destruction of property and loss of service of public utilities has not been experienced by the people of this section of country before.

The direct cause of this flood was what is called a cloudburst, and seemingly affected the Prairie River first. That river was up and over its banks before it was realized, and sweeping down on the big dam of the Grandfather Paper Mills Co. It washed away the earth and stone wing walls and thus sent a volume of water out into the Wisconsin River, which being unexpected was beyond control upon short notice. This head of water reached Merrill and before anything could be done it swept away the east end of the dam at that place, thus turning loose a second reservoir into the already fairly full channel of the Wisconsin. When this body of water reached Brokaw it very quickly breached the old, overloaded dam, releasing this time the water from a reservoir 4½ miles long and having a head of something over 20 ft. Then this three-story flood rushed down on Wausau at midnight, causing great loss of property, the principal features having been shown in Engineering News of Aug. 8, 1912. The cuts in that article are noted below.

But there is another phase of the story which may be read between the lines of all descriptions such as the above, and that story would read something like the answers to the following questions: (1) Why should dams be permitted to be built and maintained that are of such flimsy structure that their danger line is only 15 in. above their working head of water? (2) Why could not (and why should not) dams be built in such a way that floods of this nature may be cared for without destroying the dam and not allowing the entire head to escape, to the destruction of all below it? There are a few such dams in the valley. (3) Why should a dam be built with scarcely enough spillway to accommodate the low-stage flow of a stream? (4) And why do men recognize the necessity of building huge bulkwarks in the middle of a stream and then leave the shore anchorage to care for itself in a loose rolling sand bank?

Since these very things are done by those who reap the benefits of the water stored behind dams of this kind, which are jeopardizing the lives and property of others, there should be an alert authority to prohibit such actions.

Emergency Repairs after the Flood

The repairs of damages are going on as rapidly as might be expected. The worst trouble encountered was the getting of timber to build cribs for breakwaters and cofferdams. The mills and lumber yards of the city are principally on islands, and no bridges were left by which to reach them. The tailrace of the electric power station was excavated out of nearly solid rock and this material was piled on either side. The water flooding over the island washed rock and sand into them until they were filled.

In building the necessary breakwaters and cofferdams to allow of clearing these tailraces, as shown in Fig. 3 of the article mentioned (p. 235 of Aug. 8), timbers were hauled from two miles downstream, where they were picked up along the shore and unloaded on the right (Fig. 3) and then conveyed across the channel to the island. With timbers secured in this way a temporary bridge was also built and a dump-car track constructed from Clarks Island to Plumbers Island (see map on p. 234). A steam derrick was erected which hoists skips on and off dump cars, carries them across this bridge, and dumps them on Plumber's Island.

It was desired to keep the sawmill opposite the electric power station running, so a wing dam was built from the sawmill up stream on about a 35° angle with the current, across to the opposite bank. This caused all the water in the channel to pass through the gates of the mill, and it also turned the water out of the tailraces of the power plant, which could not be entirely cut off by the head gates of the plant.

The coffer-dam necessary at the guard-lock was not begun until timber was more easily had (see Fig. 2, p. 235). The abutment shown there is being replaced by a solid concrete pier and retaining walls, in place of the old masonry. This abutment will go down to solid rock, while the old abutment went only to gravel and loose rock and was back-filled partly by wood slabs with earth on top. This slab filling is in a badly decayed condition and the earth topping it was settling, causing a sliding and increasing pressure on the back of the abutment. The contract for building this abutment was let to local men for $1980.

One span of the tannery bridge which was washed away and carried about half a mile downstream will be moved up river to the High Bridge (Fig. 1, p. 234) and there erected in place of the damaged part of the High Bridge. The moving and putting in place of this span was let to the Wausau Iron Works for $750. At the tannery bridge site, this firm will put in two sets of tubular piers and build a concrete abutment in an average depth of 26 ft. of rapid water, and will furnish two steel spans of 80 ft. and one of 70 ft.; these will be 16 ft. wide and have plank floors. This work will cost $14,900.

The construction of the dam to close the gap shown in Fig. 3 (p. 235) is progressing very rapidly; two of the three piers are practically completed. There will be one needle gate (at the right of the view) and two Taintor gates next to it, giving this total amount of gate room more than there was in the old dam, or some 80 ft.

The railways have been greatly hampered by the sinking and partial washing away of the filling which they did after the flood. Sand and gravel was hauled in by train loads to make the fill where tracks were washed away (as shown in Fig. 6, p. 237). This fill was 100x200 ft. and 10 ft. deep. After it was completed, another storm of considerable consequence came up, and on Aug. 9, 10 and 11 the river was so high that it took out a large portion of this fill and caused such settlement of the remainder that two box cars standing on the track fell over on their sides. This kept all crews busy for three days strengthening and trying to save what had already been done. Trains are again hauling in gravel to complete the fill.

Trains on the main lines have just got back on their schedule. The C., M. & St. P. Ry. has had a small army of men working between Tomahawk and Wausau for the last 20 days repairing the damaged tracks. The bridge used jointly by the C., M. & St. P. and the C. & N. W. Ry. (shown in Fig. 5, p. 236), is being replaced by a temporary trestle.

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Panama Canal Excavation remaining to be done after July 31, 1912, is estimated to be 36,235,948 cu.yd., which gives a total excavation of 212,227,000 cu.yd. for the entire canal from the time of the American occupation until completion, or some 17 millions of yards more than the estimate of a year ago.

Coefficients of Flow in Wachusett Aqueduct

By James A. Cushman*

During the summer of 1911, there was installed at the Wachusett Dam (of the Metropolitan Water-works of Massachusetts) a hydro-electric power plant, designed to utilize the water flowing into the Wachusett Aqueduct. As a part of the tests and ratings of this plant, measurements of the quantity of water flowing were taken, which made it possible to establish definitely the hydraulic elements and the curve of flow in this aqueduct. It is the purpose of this paper to announce the results of these measurements, and to propose formulas deduced therefrom.

The Wachusett Aqueduct is a horseshoe-shaped conduit of brick and concrete, built in 1897-1898 as a part of the Metropolitan system, to convey the waters of the Wachusett Reservoir on the south branch of the Nashua River to the Sudbury Reservoir in Southborough. It is partly tunnel, partly a cut-and-cover structure, and partly open channel. The point where the measurements of flow were taken is in a manhole in the cut-and-cover section, 2½ mi. below the dam. The aqueduct is straight for 1270 ft. above this gaging manhole and continues in the same line for 22 ft. after passing it. The grade, 1 in 2500, is the same from a point 3400 ft. above it to a point nearly 34,000 ft. below, at the terminal chamber of the covered aqueduct. The principal dimensions of the section are given in the accompanying sketch, Fig. 1. Attention is particularly called to the fact that the conduit is brick-lined to a point 5.4 ft. above the invert, and is of concrete, plastered, the rest of the way over the arch. This factor is of very great importance in the calculations of formulas and the plotting of curves of flow.

Previous to the installation of the power plant, the quantity of water flowing was measured by a weir at the terminal chamber, in turn rated from time to time by current-meter measurements at the gaging manhole. The ordinary weir formulas were not used because of the great velocity of approach, and its attendant uncertainty. At the time the power plant was installed, special Venturi meters (an account of which appeared in Engineering News, Feb. 15, 1912, p. 294) were designed and constructed to measure the water passed into the aqueduct, taking the place of the weir.

With this brief history and description, the reasoning and the calculations of flow will be readily followed.

In calculating the flow of water in open channels and conduits, the important factor is the nature of the surface in contact with the water, i.e., whether it is rough or smooth, clean or foul, and various observers have made experiments in the past, in order to establish definite values of this "coefficient of roughness" to apply to the flows in conduits made of brick, plank, concrete, etc. Probably the most elaborate tabulation of such values was made by Ganguillet and Kutter. The Kutter formula for coefficient of flow, which takes into account the coefficient of roughness, has been used in these calculations. This formula in English units as used is

$$C = \frac{\frac{1.811}{n} + 41.65 + \frac{0.00281}{s}}{1 + \frac{n}{\sqrt{r}}\left(41.65 + \frac{0.00281}{s}\right)}$$

where

C = Coefficient of flow,
n = Coefficient of roughness,
s = Slope,
r = Hydraulic radius of the section.

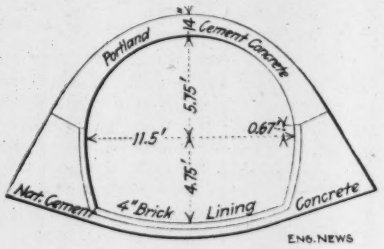

Fig. 1. Typical Cross-section, Wachusett Aqueduct

In conduits where the lining is wholly of the same material, the coefficient of roughness, n, is constant for all depths, and the curve of the same is a straight line. But in the Wachusett Aqueduct the value of n should be constant below the top of the brick lining, and should become less above that point, approaching the value for concrete. Moreover, as this coefficient is influenced solely by the nature of the wetted surface, it should vary more or less closely with the ratio of the brick lining to the plastered concrete. The first step, then, is to establish a curve of n. This was done as shown in Fig. 2.

A curve was plotted of the ratio of the brick-lined wetted perimeter to the concrete wetted perimeter against depths of water. The ordinate is infinity at 5.4 ft. above the invert where the concrete wetted perimeter is zero. Then from three ratings which were available and considered comparable, the values of n were calculated and plotted, and the curve just established was moved bodily until it passed through these three points. This curve gave values of n from the top down to a point 1 ft. above the brick lining. Several observations while the flow was entirely below the top of the brick lining, made in early ratings of the weir, were then used to derive values of n for the brick-lined section. These values were plotted and an average straight line drawn to represent n below the top of the brick lining; this curve should be a straight line because the coefficient of roughness should be constant at all depths against the same kind of lining. The two curves were then connected by an irregular curve tangent at each end to them.

There is an additional factor to be considered, which has not to this point been mentioned: that of the foulness of the wetted surface. It was realized at the very beginning of the calculations that the various ratings and observations used must represent as closely as possible the same degree of foulness in order that the results may be comparable.

Once each year, beginning with 1899 and continuing to 1906 (with the exception of 1900) the aqueduct was thoroughly scrubbed with wire brushes to remove all organic growth. Since that time it was cleaned in November, 1908, and again in April, 1911. In using early ratings care was taken to use only those which were made at the same times relative to the cleanings. The following table gives various information available in regard to the ratings and observations which were used to get values of n for the curve below the top of the brick lining.

Rating Date	Aqueduct last cleaned	Days in use since cleaning	Q flowing c.f.s.	In.
Apr. 20, 1899	Apr. 5, 1899	15	178.5	0.01239
Apr. 21, 1899	Apr. 5, 1899	16	145.2	0.01238
May 15, 1899	Apr. 5, 1899	40	113.9	0.01234
May 17, 1900	Apr. 5, 1899	401	89.6	0.01269
May 17, 1900	Apr. 5, 1899	401	92.4	0.01288
May 9, 1901	Apr. 14, 1901	91	85.6	0.01181
Apr. 26, 1907	Dec. 21, 1906	122	220.3	0.0120
Water above top of brick lining				
Mar. 24, 1904	Dec. 8, 1903	66	350.5	0.01216
Feb. 13, 1912	Apr. 15, 1911	219	492.7	0.01153
Mar. 13, 1912	Apr. 15, 1911	250	428.4	0.01171

With the exception of two ratings in 1900 and one in 1907 the observations were made soon after a cleaning. These three observations have been purposely included in the averaging, because the average time elapsing between the cleaning and the three ratings used to get values of n above the brick lining was about six months, or in other words the plotted points correspond to aqueduct only moderately clean. It is hard to say just how far to go in this discrimination, however, because above a certain point, 7.2 ft. above the invert, the aqueduct had never been wet before the date of the large flow in 1912, or in other words it was practically a new aqueduct. But it is believed that the value of n just derived is as nearly true to the condition representing a moderately clean aqueduct as any.

Having, then, a curve of n, values of C, the coefficient of flow, were calculated by Kutter's formula and a curve plotted.

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This curve naturally shows an abnormal condition for about 1½ ft. above the top of the brick lining, due to the values of n changing radically, but smooths out again as the ratio of concrete to brick lining increases more and more slowly, influencing the value of n correspondingly.

With values of C taken from its curve, various quantities were calculated by the Chezy formula, $Q = ca\sqrt{rs}$, and a curve of flow plotted. This curve shows that the maximum flow occurs at about 9.75 ft. above the invert, or in other words when the aqueduct is about 93% full. About 97.5% of the available area is used, and the maximum flow is expected to amount to 363,000,000 gal. per 24 hr., or 562 cu.ft. per sec.

The aqueduct was designed to have a capacity of at least 300,000,000 gal. per 24 hr., and immediately after its completion, as a result of the original ratings, it was estimated that the maximum capacity would be about 321,000,000 gal. per day. The value of n used in this estimation was 0.0124 for a clean brick-lined aqueduct, and the value of C computed was 141. It seems to be reasonable to credit the lower values of n now used, and the consequent increased carrying capacity, to the manner in which the aqueduct has been cleaned (scrubbing with wire brooms undoubtedly rubs off the small rough projections of mortar from the edges of the joints, removes any particles adhering to the face of the bricks, and rounds off the sharp corners and edges of the brickwork), as well as to the cement lining of the upper part of the section.

The values of n derived were: For the brick-lined surface, 0.0123, and for the combination of brick and concrete surface, 0.01135. Ganguillet and Kutter give for new well-laid brickwork, 0.011 to 0.012, and for cement (one-third sand), 0.011. It seems reasonable to assume that, had this conduit been built entirely of concrete as smooth as the upper part was built, the value of n could have been taken as low as 0.0112, and perhaps lower, at the point of maximum flow.

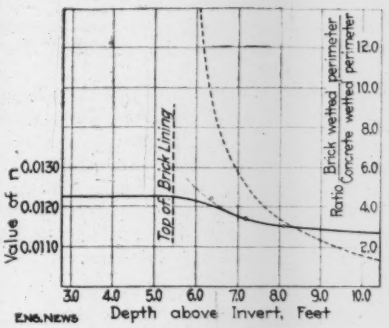

FIG. 2. DERIVING COEFFICIENT OF ROUGHNESS FOR VARIOUS DEPTHS IN WACHUSETT AQUEDUCT

The values of C derived and the quantities flowing were as follows:

	C	Q
Aqueduct ¼ full	136	61 m.g.d.
Aqueduct ½ full	144	156 m.g.d.
Aqueduct ¾ full	154	271 m.g.d.
Aqueduct at max. capacity	158	363 m.g.d.
Aqueduct full	157	349 m.g.d.

If the aqueduct had been built entirely of concrete and the value of $n = 0.0112$, assumed as given above, the value of C would be 161 at the point of maximum flow, and the flow itself would be 570 cu.ft. per sec., or 370,000,000 gal. per day, an increase of about 2% in carrying capacity.

Following the investigations and ratings in the Sudbury, Stoney Brook and Croton aqueducts, there were derived the following three formulas for the value of C, respectively: $127\ r^{0.12}$, $122.6\ r^{0.07}$, and $124\ r^{0.06}$. With these as precedents the writer ventures to suggest similar formulas for the Wachusett Aqueduct. In this structure it was not possible to devise one formula which could be expected to give close values of C for all depths, on account of the varying n above the depth of 5.4 ft.; therefore, two formulas have been derived, one for all depths up to the top of the brickwork, and the second for depths above that point. The first is similar in form to the three above quoted, and was probably derived in the same manner, i.e., by assuming one constant in the formula $C = ar^b$, and solving for the other, using various values of r until the error in the calculated C was reduced to a minimum. The formula finally reached was $C = 124.5\ r^{0.15}$. Using this constant and index, values of C were calculated which varied from the curve originally plotted, from 1% to 3% for depths below 2 ft. above the invert, and less than 1% for depths between this point and the top of the brick lining.

For flows above the brick lining the value of n varies with the depth, and consequently it must be included in the formula. As it varies inversely as C, the formula will take the form $C = \frac{ar^b}{n}$. Assuming one constant and solving for the other, in the same manner as in the first formula, will give finally both. In this second formula, the index b was assumed the same as in the first formula, 0.15, and the other constant was found to be 1.515. This formula, $C = \frac{1.515\ r^{0.15}}{n}$, gave values of C which varied less than 1% from the curve values for all depths above the brick lining.

It is therefore believed that these two formulas will give working values for the coefficient of flow applicable to conduits of the shape and construction of the Wachusett Aqueduct. It must, of course, be remembered that the values derived are too large for a foul aqueduct, and may be from 5% to 10% in error, varying with the degree of foulness. But if the conduit is cleaned reasonably often, the discharge at any depth should agree closely with the calculated quantity.

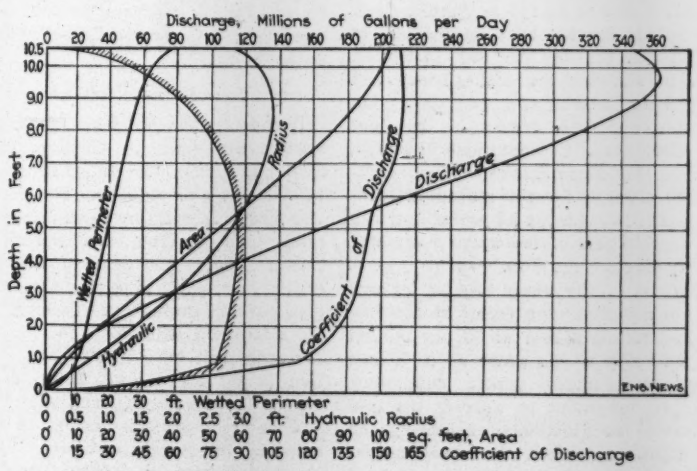
Discharge, Millions of Gallons per Day
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360
Depth in Feet
10.5 10.0 9.0 8.0 7.0 6.0 5.0 4.0 3.0 2.0 1.0 0
Wetted Perimeter
Area
Hydraulic
Radius
Coefficient
of
Discharge
Discharge
ENG. NEWS
0 10 20 30 ft. Wetted Perimeter
0 0.5 1.0 1.5 2.0 2.5 3.0 ft. Hydraulic Radius
0 10 20 30 40 50 60 70 80 90 100 sq. feet, Area
0 15 30 45 60 75 90 105 120 135 150 165 Coefficient of Discharge

FIG. 3. HYDRAULIC ELEMENTS OF WACHUSETT AQUEDUCT, FROM RATINGS

Road Irrigation for Dust Prevention is practiced in the Imperial Valley, Southern California. A low ridge in the middle of the roadway and an embankment on either side convert the roads into two shallow ditches. These are flooded alternately from irrigation ditches which convey Colorado River water to the adjacent cultivated land. While one-half of the roadway is flooded, the traffic takes the other side. There is practically no rain in the Imperial Valley, portions of which are more than 200 ft. below sea level. This method of dust reduction is said to be low in cost. It is also said that with the present population and taxable values the cost of oiling the roads would be prohibitive.

Improvements in Pittsburgh's Water-Supply System

The Pittsburgh filtration plant, built on the north bank of the Allegheny River at Aspinwall seven years ago, has since its construction supplied all of the main portion of old Pittsburgh* and part of the South Side. Improvements are now under way which will extend the supply of filtered water to the North Side (Allegheny). They include (1) the construction of a large pumping station at the filter plant, (2) the construction of a preliminary treatment plant to make the sand filters work more economically, (3) the putting in service of a bank of additional filter beds, built two years ago but not yet used because of leakage through the floor, (4) the construction of a new distributing reservoir for the low-service district in Allegheny. An auxiliary improvement, though not related to the supply of Allegheny, is (5) the construction of a new pumping station on the South Side for the high-service territory of that district.

Adoption of Preliminary Treatment

The operation of the filtration plant has been satisfactory in point of quality, but its economy has not been all that could be desired. The length of run of the filter beds between cleanings is highly variable, and at certain times is so short as to make the cost of the water abnormally high. In the worst periods the output per bed between cleanings is as low as 30,000,000 gal. The trouble appears to come chiefly from colloidal suspended matter in the river water, and is most serious in times when the water is quite clear. Coal-mine wastes discharged into the river are believed to be responsible for the colloidal matter which coagulates in the filters and clogs them. With a view to improving the action of the filter beds, a series of 24 "contact baffles" and three A-frame baffles is being constructed in Basin B.

Each of the "contact baffles" or gravel filters is a concrete chamber 40x60 ft., to carry a 9-ft. depth of gravel of ½ to 1 in. size, which rests on a grating of separately-cast reinforced-concrete beams. The gravel is thus underdrained. Cleaning will be done by flushing, under extra head, without reversal of flow.

The contract for the construction of these preliminary beds was let to the Duquesne Construction Co., completion being due Jan. 1, 1913. The contract price is $155,000.

Water Disinfection—The preliminary treatment is designed principally to lessen the cost of operating the filters, and to increase the total capacity of the plant, by lengthening the runs between cleanings, so that the rate of filtration may be raised. Chemical coagulation has not been used up to the present time. The character of the water is such at times that this would be distinctly advantageous in connection with the proposed pretreatment.

However, disinfection by calcium hypochlorite has been used for some three years past as a supplementary means of rendering the water safe against the transmission of water-borne diseases. During the past year disinfection has been in use continuously. Prior to that time it was used only intermittently.

Waterproofing New Filter Beds

Three years ago the Bureau of Filtration (since then disbanded) constructed a series of ten additional filter beds. These are located close to the river and are on sandy ground. Their design differed in some important respects from the design of the original filter beds, though the two series of beds were designed by the same authorities. In the old beds the columns supporting the roof rested on a groined-arch floor, the invert being 8 in. thick at the crown. The new beds, however, had their posts carried to independent pier footings, 5 ft. square, and the floor was formed by a 5-in. flat slab at the level of the column bases. When filtration tests were made with the new beds it was noted that there was a loss of head even when the outlet gates were closed. An analysis of the situation suggested that the floor-slab might have cracked, which inference was borne out by subsequent examination.

The leakage of water was measured, and was found to be 1½ to 2 million gal. per day for the ten beds, or 100,000 to 220,000 gal. per day per bed, in winter, while in summer the leakage per bed dropped to 20,000 gal. per day. On emptying one bed, cracks in the floor were discovered. Computations indicated, however, that these cracks were not sufficient to account for the measured leakage, and it is regarded as probable that a considerable part of the water percolated directly through the 5-in. slab.

The cost of the water lost by leakage was not a determining factor in deciding on repairs. At a cost of $6 per million gal., the value of the water which leaked away, even under winter conditions, would be too small to warrant the cost (estimated at $150,000) of making the floor tight. However, the sandy nature of the subsoil and the difference of head between the filter beds and the river (about 25 ft., the river being close to the bed) gave reason for apprehending that the continued seepage might tend to wash out channels in the subsoil and endanger the stability of the structure.

Plans were therefore made for waterproofing the beds, and a contract was let to John F. Casey Co. for $135,000, the work to be completed Jan. 1, 1913. This waterproofing consists of placing two layers of felt between three layers of asphalt on the old floor and covering with a 4-in. layer of concrete laid close against the columns. Measurement of leakage in those beds which have already been waterproofed by this treatment shows 600 gal. per day per bed, which is within the errors of measurement.

Aspinwall Pumping Station

A 100,000,000-gal. pumping station is being built at the filtration plant for the new service to Allegheny. The present Allegheny supply comes from a point on the river several miles above the filter plant through a force main which passes near the filter plant. The new service will be delivered through this same main by a branch from the new pumping station, and the direct river supply will then be discontinued.

The new pumping station will ultimately have five 20,000,000-gal. pumps, which will not only give capacity for supplying Allegheny but will also permit carrying a pipe line under the Allegheny River in the lower part of the city to supplement the supply to the downtown low-service district of Pittsburgh. The cost of the new pumping station, complete with five pumps, will be about $1,100,000. The pumps, built by the Allis-Chalmers Co., are vertical triple-expansion pumping engines. They are guaranteed to deliver 196,000,000 ft.-lb. per 1000 lb. steam.

The superstructure of the pumping station is about to be placed under contract. The foundation, now nearly completed, was built by the John F. Casey Co. The boilers for the new station will be supplied by the E. Keeler Co., of Williamsport, Penn.

New Storage Reservoir

A contract has just been let for a new storage reservoir on Cabbage Hill in Shaler township, near Allegheny, to be of 150,000,000-gal. capacity. This reservoir will be of the ordinary earth-embankment type, with concrete and waterproofing lining. Supply and delivery will be controlled by a single gate-chamber, the inlet pipe passing through this chamber to the far side of the reservoir. The cost of the reservoir will be $620,000, and completion is set for Jan. 1, 1914. John F. Casey Co. are the contractors.

This reservoir will correspond to the existing Pittsburgh reservoir Highland No. 2, both being at an elevation of about 275 ft. city datum. This corresponds to the lowest of the city's three water-pressure zones (the other two being supplied from Highland No. 1 and Herron Hill reservoirs). Interconnection of the Allegheny and Pittsburgh water systems by a main under the Allegheny River will therefore be possible.

*Except a few strips of territory in the eastern part, supplied by private companies operating mainly outside of the city area.

Highland No. 2 reservoir incidentally is of interest as having developed very great leakage through defective work in lining it, at the time of its construction. Investigation at that time showed that the reservoir was apparently in no immediate danger from the leakage, and that the loss of water, though great, did not warrant undertaking repairs at that time. Since then the loss has increased and it is now nearly large enough to make repairs an economic necessity; there is apparently no structural danger to the reservoir, however.

South Side Pumping Station

A pumping station of 28,000,000 gal. ultimate capacity has been built in the South Side district and is now nearly completed. It should be in service before winter. This station is located on Mission St. It is designed to contain four 7,000,000-gal. vertical triple-expansion pumping engines, built by the Bethlehem Steel Co. Only two are being installed at the present time. The engine room is so arranged that the entire machine from the foundation of the pump end to the top will be exposed, there being no intermediate floor.

This pumping station will supply the high service district of the South Side, in place of a service zone now supplied by the Monongahela Water Co. The head pumped against will be about 220 lb. The cost of the station is about $500,000.

Including all the several improvements above mentioned, the Water Bureau has about $3,000,000 worth of work now under construction. The Bureau designs all its own work. C. A. Finley is superintendent of the Bureau of Water, and the above data concerning the water supply are based on information supplied by him.

Examination for Assistant Engineer: State of New York

By A Subscriber

The following is a list of questions and instructions for the examination for assistant engineer, held by the New York State Civil Service Commission, July 27, 1912, as nearly as the questions can be remembered.

PART 1

INSTRUCTIONS—The examination will consist of 14 exercises on two sheets, of total weight 200. Of these, it is required that competitors take, on the first sheet, sufficient to make up a total of 100 for both sheets. If more than this is taken, only those exercises, amounting to the required total weight, which obtain the highest credit, will receive consideration in fixing the average rating. The weight of the selections made may unavoidably overrun 100 by 5%, in which case the weight of that one receiving the lowest percentage rating will be reduced to make the total 100.

The use of a slide-rule and logarithms is advised. Any reference books may be used on Part 1 of the examination.

If the statement of conditions of any problems seems inadequate, the competitors may assume any further data not inconsistent with those already given. Incomplete designs and computations will receive credit in proportion to their value, in the opinion of the examiner, as part of the whole work required. Direct methods and orderly arrangement will have weight in fixing ratings.

Select problems in Part 1 of total weight of 60.

(1) In putting up a building an 80-ft. derrick tower 15x15 ft. is to be used, carrying two booms, each 50 ft. long, set on foot steps 45 ft. above ground, at diagonally opposite corners of the towers. Each is expected to swing over an arc of 270° and to carry three tons in any position.

Figuring upon a wind load of 30 lb. per sq. ft. of exposed area, compute the stresses in all the members of the tower, assuming it built of timber. Make sketches of the tower, showing particularly details at supports of booms, hoisting lines and base. (Weight 20)

(2) A single track railway runs on a tangent from Sta. 550 to 625; on a 3° 30′ curve from 625+00 to 639+80, skirting around a hill of slate rock with side slopes of 1 on 2½; and on tangent from 639+80 to 708+00.

A canal, built on tangent making an angle of 40° 00′ with a tangent produced from 628+00 and whose center line is 120 ft. from the nearest point of center line of the track, is to be moved toward the railway 80 ft., requiring a corresponding shift in the track.

Assuming the land level outside of the hill mentioned, sketch a design for a rearrangement of the track, fixing the beginning and ending of the new curve, and compute the quantity of the principal items in the work. (Weight 15)

(3) A concrete retaining wall 10 ft. high, 2 ft. thick at the top and 4.5 ft. thick at the bottom, resting on a rock foundation and supporting a clay bank level with its top, is to be raised 6 ft. to hold up a fill of gravel 14 ft. high over the clay, making a surcharge of 8 ft.

The conditions require that the old wall remain, and the addition be on the lower side, but may not encroach more than 1.5 ft. in this direction, except that, below the level of the bottom of the old wall, the footing may extend as far as desired.

Design the section for the remodeled wall, showing a full computation of the lateral pressures, stability of the wall, including the amount and disposition of the reinforcement, etc. Include also a detailed sketch and description showing the method of constructing. (Weight 25)

(4) A rectangular gate-house, 15 by 25 ft. inside, with walls 3 ft. thick, carried up to Elev. 450, admits water over the flat tops of the walls and through three sluice gates 2 ft. 6 in. wide by 3 ft. 6 in. high, the centers of these openings being at Elev. 435, 420 and 400 respectively.

Leading from the gate house with its center at Elev. 400 is a circular brick culvert 7 ft. in diameter, 300 ft. long built on a 2½% slope, with free discharge at its lower end. Compute the discharge from the culvert when the reservoir is at Elev. 451, all gates being open. (Weight 20)

(5) A wooden pipe carrying water under a head of 80 ft. is made up of staves 1½ in. thick. Find the size and spacing of round iron bands to hold the pipe together. (Weight 10)

(6) Design and compute the stresses in a wooden form for a plain concrete overflow dam 20 ft. high. Make sketches of the details. (Weight 10)

(7) In fixing on the ground the line of conduit ACB, linear measurement from A to B was found impracticable. A transit line was run, with a deflection at C of 2° 20′ to the right. A base line DE, 1002.71 ft. long, and the following angles were measured: CAE, 81° 20′; DAE, 36° 15′ 30″; DEA 103° 58′ 00″; DEC, 53° 20′ 00″; ADE 39° 46′ 40″ DBC, 44° 30′ 00″; all angles being read on the same side of ACB. Compute the length of distance AB. (Weight 15)

PART 2

Answer questions of total weight 40 sufficient to make 100 added to those taken on Part 1. No reference books are to be used.

(8) Discuss the following situation: A contractor disputing with an engineer the validity of an order requiring him to put in, in a certain way, some foundation work, which will later be covered up, is directed to discontinue all work. He, however, leaving the part in question, takes up and puts in, according to the specifications, other sections of the contract. On arbitration, the question in dispute is decided against the contractor, the owner having meanwhile completed this work at a cost somewhat above the price bid for it. Is the contractor entitled to payment at the face bid for the work done contrary to orders? Mention the conditions that would be likely to influence a decision in such a case. (Weight 10)

(9) Describe in detail the sequence of operations in drilling, loading and firing, and mucking out a 7 by 10 ft. tunnel, driven through limestone. An explanation of the practical difficulties and the precautions necessary, and the details of methods, rather than a description of general problems, should be given. (Weight 15)

(10) Compare the merits of cast iron, riveted steel and reinforced concrete for long pipe lines. (Weight 10)

(11) What practices of a road building contractor would give him larger estimates for grading and surfacing than the quantities actually put in? How would the form of the contract influence tendencies in this direction (Weight 10)

(12) Near the head of a series of rapids in a river 700 ft. wide and 4 ft. deep, a curved dam with radius of upstream face 1,100 ft., is to be put in. The center of the curve (circular throughout) is 200 ft. out in stream from one shore.

Explain by sketch and description how lines and grades would be given, both during the preparation of the foundation, and during construction, showing the location and manner of establishing working and reference points.

Assume that the exact position is fixed with respect to a property line a quarter of a mile down stream. How large a party will be needed for this work and for what time? (Weight 15)

(13) What features of a watershed effect the run-off of streams? Assume the case of a creek 10 miles long, draining an area of 50 square miles, and explain how the average and maximum and minimum flow would be estimated, if only a description of the country were available. (Weight 10)

(14) Describe what you consider the best method of filing and indexing construction drawings in an office handling a number of separate projects. (Weight 10)

A Butting Collision between a passenger train and two freight locomotives occurred on Aug. 18 near Troy, Ohio, on the line of the Cincinnati, Hamilton & Dayton Ry. All the passenger coaches were derailed but there were no serious injuries to passengers. The engineer and the fireman of the first freight engine were badly hurt. At the time of the accident the passenger train had the right of way.

A Derailment Attributed to a Defective car wheel occurred on the Boston & Maine R.R., near Lakeport, N. H., on Aug. 19. An express train was rounding a curve at 25 miles an hour when it left the rails. Two members of the crew were injured.

A Grandstand Collapsed at Indianapolis, Ind., on Aug. 20, during the ceremony of notifying Governor Marshall of his nomination as Democratic candidate for Vice-President. The stand was a temporary wooden structure and there were on it, at the time of the accident, about 500 spectators, a large number of them being women. More than 100 of these people were injured, none seriously. The failure was not sudden, but was preceded by a swaying motion from side to side, after which the center portion of the stand collapsed.

The Lackawanna Wreck at Corning, N. Y., on July 4, is attributed, in the report of the Interstate Commerce Commission, principally to the negligence of Engineer Schroeder in ignoring automatic block signals and signals given by a flagman. The report states also, that the single steel car involved in the wreck was comparatively little damaged, and suggests that increased safety would be secured (1) by making flagging rules more explicit, and requiring the use of

torpedoes, and (2) by more proper overlapping of automatic block signals, and by either reducing the speed of fast trains in foggy or stormy weather, or by installing automatic train control devices.

A Derailment Due to a Block Signal having been struck by lighting took place on the Chicago, Milwaukee & St. Paul Ry., near Grayland, Ill., on Aug. 21. The signal controlled the closing of a switch, and when it was disturbed the switch failed to close. The train struck this open switch at a speed of about 30 miles an hour, and the locomotive and baggage cars were ditched, injuring the engineer and the fireman.

The Waste in Coal Mining, including the portion rendered unfit for use, and that left in the mines in such shape that it is hard to recover, is claimed to be about 50% of the total useful output of the mines. This statement is made by J. A. Holmes, Director of the U. S. Bureau of Mines, in a preface to a bulletin entitled, "Notes on Mineral Waste," by C. L. Parsons.

A Freight Wreck, involving considerable loss of property but no casualties, occurred near York, Ont., on the line of the Grand Trunk Ry. on Aug. 20. A heavy double-header freight train, running on a down grade, collided with a stationary freight train and resulted in the disabling of two locomotives and the destruction of six freight cars. A fire broke out in the wrecked cars, adding to the damage. The track was blocked for three hours.

An Explosion and Fire at the plant of the Sun Oil Co., Toledo, Ohio, on Aug. 20, caused the death of one employee and the injury of three others. According to reports, the accident was due to escaping gas, from a tank-car of gasoline, coming in contact with a fire in the boiler room, 75 ft. away. After the explosion, the flames jumped back to the car of gasoline and to a still containing gas, and the latter also exploded, scattering débris around the plant and adjacent property. The total loss is estimated at $15,000.

Another Trespassing Accident occurred on the line of the Baltimore & Ohio R.R., near Frostburg, Md., on Aug. 18. Four persons were killed and two injured, all of them women. A picnic party, consisting of these six women and one man were walking toward Frostburg along the railroad tracks and had just left one track to escape injury from a westbound freight, when a fast passenger train, eastbound, traveling 30 miles an hour, bore down upon them. The accident happened on a curve, and this fact, as well as the fact that the passenger train was coming from behind them, served to keep the party in ignorance of their danger until too late to escape.

A Fire in the General Post Office buildings in London, Eng., on Aug. 24, thoroughly crippled the telegraphic communication to the city by destroying all the main wires in the building, which is the center of the telegraph service for London, and, in fact, for the entire Kingdom. In consequence all communications from London to this country were very much delayed and a full account of the fire had not reached here before we went to press. It is known that seven international cables centered in the building, and all communication through them has been cut off and will be cut off for probably a week from the time of the fire. The building is of reinforced concrete, and probably the largest structure of that type in Great Britain. So far as can be learned, the fire started in the telegraph section among the wires and did not reach that portion of the building where mail matter was kept. The only damage appears to be in the wiring and the destruction of some furniture, and all the employees had ample time to leave their posts. Apparently the reinforced-concrete construction served to retard the progress of the fire and to save the building from destruction, but authentic reports are not yet available.

Lock Gate Collisions a Habit—The several collisions of steamers with lock gates in the Welland Canal seem to have become so much a habit that they are creating but little excitement in the neighborhood of the canal. On Aug. 16 the steamer "Harry E. Packer" passing up the canal struck the headgates of Lock 24, and damaged them sufficiently to interrupt navigation for several hours. Again the accident resulted from a failure of the boat to respond to the captain's signals. In this instance, it is claimed, the engineer misunderstood the stop signal for a go-ahead signal, breaking the snubbing lines and running into the headgates. New gates were at once ordered to be brought from Port Dalhousie.

A Bronze Bust of Joseph Henry, the distinguished scientist, has been presented to the American Institute of Electrical Engineers by Past-President Gano Dunn. This bust is taken from the life-size statue of Henry, by Herbert Adams, in the Smithsonian Institution at Washington, D. C., and it was reproduced under the supervision of Mr. Adams.

The Imhoff Tank Patents in Germany have been upheld by a decision of the "Reichsgericht," the highest tribunal in Germany. The decision ends a suit for the nullification of the Imhoff patent brought by Messrs. W. O. Travis and E. Ault, of Great Britain, who claimed that the Imhoff patents infringed those already issued for the so called "Travis tank." The Imperial German Patent Office, on May 18, 1911, in the first consideration of the suit, restricted the Imhoff patent to its claim on the apparatus alone, and even modified that claim, as originally allowed. On appeal the German Court has decided that the original Imhoff patent is valid in both process and apparatus, and that it does not infringe the Travis patent. Full costs of the action were assessed on Messrs. Travis and Ault.

A Pitot-Tube Meter with a helical gage tube is being put out by the Sargent Steam Meter Co., of Chicago, for measuring the volume of air or gas flowing through a pipe. The meter consists of a wide tube, or well, around the circumference of which is a semi-circular groove in which is wound a transparent celluloid tube, the lower end of which connects to the well and the upper end to a passage, through the yoke of the well frame, to the tube in the pipe whose opening is perpendicular to the flow of gas or air. The well is connected by a similar opening to the other tube having one or more openings facing the flow of air or gas. Water or oil is put into the well through an opening made for the purpose until it reaches a zero mark on the helix, with the liquid surfaces in the well and helix open to the atmosphere. When the pitot nozzle is connected up and gas or air is flowing through the pipe, the resultant pressure is transmitted to the well and the liquid in the helix rises in proportion. For each inch of vertical rise the liquid travels about 10 in. around the well, and as this distance is graduated into 100 parts, the head can be read to 0.01 in. The well and helix are revolvable on the yoke, so that the head can always be read without changing the connections to the pipe.

Heating Trains with the Exhaust steam of the air brake pump is made practicable, it is claimed, by a new device of which the essential feature is a special valve which relieves the back pressure on the air-pump cylinder at the point when the pressure of the air being compressed plus the back pressure is equal to the air pressure in the main reservoir. As long as the pressure of the compressor exhaust is higher than the steam pressure carried in the heating system, the valve will connect the compressor exhaust to the heating system; but as soon as the falling pressure of the compressor exhaust equalizes with the rising pressure in the heating-system reservoir, the special valves automatically operate to cut off the heating system and to open the compressor exhaust to the atmosphere.

It is claimed that the operation of this valve is such that very little difference in the back pressure can be noted in the operation of the compressor with the valve and without it, and that 83½% of the total steam used to operate the compressor can be diverted by this method to the heating system.

The somewhat complicated mechanism of the valve, which is now being put on the market by the Ward Equipment Co., of 141 Cedar St., New York City, under the trade name of the "Econotherm," is described in detail in an article published in the July 13 issue of "The Railway & Engineering Review."

Personals

Mr. Arthur Squires has been appointed Chief Mechanical Engineer of the Chilean State Rys., with headquarters at Santiago, Chile.

Mr. Marsden Manson, M. Am. Soc. C. E. has resigned as City Engineer of San Francisco, Calif., after many years of public service. Mr. Manson has been ill for some months.

Mr. J. S. Reddock, formerly Superintendent of the National Rys. of Mexico, at Aguascalientes, Mex., is now Assistant Superintendent of the Missouri & North Arkansas R.R., at Leslie, Ark.

Mr. J. M. Scott, formerly Chief Inspector for the Indiana Railway Commission, has been appointed Supervisor of Transportation of the Baltimore & Ohio R.R., with headquarters at Baltimore, Md.

Mr. W. R. Hudson, until recently General Superintendent of the Norfolk Southern R.R., has been appointed Assistant General Manager of the Atlanta, Birmingham & Atlantic R.R., with office at Atlanta, Ga.

Mr. J. W. Sneed has resigned as Superintendent of Construction, Chagres district, Panama Canal, to engage in the promotion of an engineering and contracting company, with headquarters on the Isthmus.

Mr. George F. Comstock, formerly Metallographist at the Park works of the Crucible Steel Co. of America, Pittsburgh, Penn., is now in charge of the physical testing laboratory of the Titanium Alloys Manufacturing Co., Niagara Falls, N. Y.

Mr. R. J. McComb, formerly Roadmaster of the Wheeling & Lake Erie R.R., at Dillonvale, Ohio, has been promoted to be Division Engineer at Canton, Ohio, succeeding Mr. C. A. Stelle, whose resignation was noted in our columns of last week.

Mr. Wm. A. Gravell, formerly a consulting civil engineer, of Philadelphia, Penn., is now associated with Franklin & Co., 1420 Chestnut St., Philadelphia, an engineering firm making a specialty of design and construction in steel and reinforced concrete.

Mr. George C. Andrews, Assoc. M. Am. Soc. C. E., formerly Assistant Engineer in charge of contract No 10, New York State Barge Canal, at Fulton, N. Y., has been promoted to be Resident Engineer in charge of residencies Nos. 10B and 11, with headquarters at Lockport, N. Y.

Mr. Jerome Newman, formerly with the engineering department of the Southern Pacific Ry., has been named to be Chief Engineer of the California State Harbor Commission, succeeding Mr. A. V. Saph, Assoc. M. Am. Soc. C. E., whose resignation was noted in our columns of last week.

Mr. H. S. Lippincott, who has been in charge of the Pennsylvania R.R. demonstration farm at Bacon, Del., has been appointed Agriculturist of the Pennsylvania R.R. system with offices in Philadelphia, Penn. This is a newly created office, the aim of which will be to better the conditions of farms and farming in the territory served by this railway.

Mr. F. Lavis, M. Am. Soc. C. E., of Guerber, Lavis & Co., Consulting Engineers, New York City, has been appointed Chief Engineer of the Argentine Ry. Co., with offices at 261 Calle Reconquista, Buenos Aires, Argentine Republic. This company has taken over some existing railways and will build several new lines and extensions. Mr. Lavis retains his New York office at 50 Church St.

Mr. Stewart M. Marshall, M. Am. Soc. M. E., formerly Assistant Chief Engineer of the Cambria Steel Co., Johnstown, Penn., has been promoted to be Chief Engineer. Mr. Marshall succeeds Mr. John W. Gocher, M. Am. Soc. M. E., who has been transferred to the executive department, where he wil make exhaustive studies of plant operation with a view to improvements and increased efficiency.

Mr. Edward Betts, Assoc. M. Am. Soc. C. E., has resigned as Chief Engineer for the Hamilton County Road Commission, Chattanooga, Tenn., and opened an office at 703 James Bldg., Chattanooga, as a civil and consulting engineer. Before holding his present office Mr. Betts was, for 21 years, engineer of the Chickamauga and Chattanooga National Military Park, and also of the Vicksburg National Military Park.

Mr. Charles Ware, General Superintendent of the Union Pacific R.R., has been appointed Vice-President and General Manager, succeeding Mr. W. B. Scott, whose election as President of the Southern Pacific Ry. lines in the Southwest is noted elsewhere in these columns. Mr. Ware entered the railway service as a telegraph operator with the Chicago & Northwestern Ry., in 1882. He was formerly Superintendent of the Nebraska division of the Union Pacific R.R.

Mr. E. P. Bracken, recently Assistant General Manager of the Chicago, Burlington & Quincy R.R., at Chicago, Ill., has been promoted to be General Manager of the lines east of the Missouri River, with office at Chicago. Mr. Bracken began railway work with the Chicago, Burlington & Quincy R.R., in 1887, as a foreman in the track department. For 13 years he was a roadmaster and for 3 years a trainmaster. He was appointed Assistant Superintendent of the Lincoln division in 1906, and since then his rise has been rapid. Mr. Bracken is succeeded as Assistant General Manager by Mr. E. S. Koller, General Superintendent, at Galesburg, Ill.

Mr. W. B. Scott, Vice-President and General Manager of the Union Pacific R.R., at Omaha, Neb., has been elected President of the Galveston, Harrisburg & San Antonio Ry., the Houston & Texas Central R.R., the Texas & New Orleans R.R., and other subsidiaries of the Southern Pacific Ry. in the Southwest, succeeding Mr. Thornwell Fay, whose resignation was noted in our columns of last week. Mr. Scott was Superintendent of the Galveston, Harrisburg & San Antonio Ry., from 1902 to 1904, and General Superintendent of the Houston & Texas Central RR. in 1905 and 1906. Mr. Scott is a Canadian by birth and began his railway career as a messenger with the Grand Trunk Ry. in 1875.

Mr. William F. Williams, M. Am. Soc. C. E., City Engineer of New Bedford, Mass., has been appointed Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, succeeding Mr. F. W. Hodgdon, M. Am. Soc. C. E., who has been appointed Chief Engineer of the Port of Boston, Mass., as previously noted in our columns. Mr. Williams received the degrees of C. E. and M. E. from Columbia University in 1881 and 1882, respectively. From 1884 to 1890 he was engaged in private practice in Hartford and Meriden, Conn., and for three years following was Chief Engineer of the Cumberland Lands, Ltd., in charge of surveys and plans for the development of 46,000 acres of iron, timber and agricultural land in Tennessee. Since 1893 Mr. Williams has been City Engineer of New Bedford, Mass., during which time he also served as Chief Engineer for the design and construction of the New Bedford and Fairhaven bridge; he has had charge for the city of grade-crossing abolition work costing $1,000,000, and has designed and built various wharf and waterfront improvements in New Bedford Harbor. He was a member of the commission in charge of surveying and plotting the entire city and designed the $1,600,000 intercepting sewer system now under construction.

The United States Bureau of Lighthouses announces the following changes in personnel: Col. Curtis McD. Townsend, M. Am. Soc. C. E., Corps of Engineers, U. S. A., has been appointed Inspector of the 15th district, succeding Lieut.-Col. C. L. Potter, M. Am. Soc. C. E., Corps of Engineers, U. S. A.; Mr. Arthur E Arledge, Assoc. M. Am. Soc. C. E., has been promoted from the position of Superintendent to be Inspector in the 19th district; at Honolulu, H. I.; Mr. Edward L. Woodruff, M. Am. Soc. C. E., has been promoted from the position of Superintendent of the 18th district at San Francisco, Calif., to be Inspector in the 11th district, at Detroit, Mich.; Mr. Camille A. Lamy has been promoted from the position of Superintendent to be Inspector of the 9th district of Porto Rico and Guantanamo; Mr. L. M. Stoddard has been promoted from the position of Master to be Inspector of the 12th district at Chicago, Ill. The act of June 17, 1910, reorganizing the Lighthouse Service, provided for the gradual appointment of civilian inspector to take the places of Army and Naval officers, then holding office. Each district is in charge of an inspector. The changes noted complete the appointments of civilian inspectors with the exception of the three river districts, which are in charge of officers of the Corps of Engineers, U. S. A. The positions are now under the classified civil service. The desirable qualifications for a Lighthouse Inspector include a knowledge of nautical affairs and the needs of navigation and training in civil, mechanical and marine engineering. The appointments have so far been made from men already in the Lighthouse Service, or in other branches of government work requiring similar qualifications. Mr. George R. Putnam, M. Am. Soc. C. E., is Commissioner of the Bureau of Lighthouses.

Col. Joseph Ury Crawford, Consulting Engineer of the Pennsylvania R.R., will retire under the company's pension rules on Sept. 1, having reached the age of 70 years. Col. Crawford is a native of Pennsylvania, and was a member of the class of 1862 of the University of Pennsylvania, at the outbreak of the Civil War. He enlisted in the 17th Pennsylvania Regiment in April, 1861, was commissioned Second Lieutenant in the 6th New Jersey Regiment the following October, and was made Captain in June, 1862. Following the war Col. Crawford engaged in surveys for the Atlantic & Great Western R.R. In 1870 he entered the service of the Pennsylvania R.R., as Senior Assistant Engineer of the Alexandria & Fredericksburg R.R. A short time later he went West as Principal Assistant Engineer and later Engineer of the Texas & Pacific R.R. In 1878 Col. Crawford received the appointment of Consulting Engineer to the Government of Japan, and for his services there he was decorated by the Emperor with the Order of the Rising Sun. After his return to America he was engaged for some years by the late Jay Gould, in making examinations and surveys for a transcontinental railway. In 1882 Col. Crawford returned to the Pennsylvania R.R. as Chief Engineer of the Pennsylvania Schuylkill Valley R.R. Since then he has been continuously in the service of the Pennsylvania R.R., and its subsidiaries, with the exception of a short period during the Spanish-American War when he served as Consulting Engineer for the United States Government on transportation facilities in Cuba. As Chief Engineer Col. Crawford had charge of the construction of the Piedmont & Cumberland R.R., the Norfolk & Portsmouth Belt Line R.R., the New York Connecting R.R., and other subsidiaries and branch lines of the Pennsylvania R.R. Since January, 1911, he has been Consulting Engineer of the company.

Obituary

Harold Gay, Assoc. Am. Inst. E. E., Inspector for L. B. Stillwell, M. Am. Inst. E. E., Consulting Engineer, New York City, died at his home in Farmington, Conn., Aug. 26.

William F. Lockwood, M. Am. Soc. C. E., Engineer of Maintenance-of-Way of the Interborough Rapid Transit Co., New York City, died at his home in Yonkers, N. Y., Aug. 23. A biographical sketch of Mr. Lockwood will appear in a later issue.

J. Hurst Fulton, Engineer with the electrical construction department of

Walter Kidde, Engineer-Contractor, 140 Cedar St., New York City, died at his home in Brooklyn, N. Y., on Aug. 19. He was 31 years old and graduated from Stevens Institute of Technology in 1905.

William A. Waterbury, formerly Superintendent of the New York, New Haven & Hartford R.R., at New Haven, Conn., died at his home in New Haven, Aug. 19, at the age of 74 years. Mr. Waterbury began railway work in 1857 as a clerk with the Naugatuck R.R. in Connecticut.

Charles F. Fell, Assistant Engineer of the Bureau of Engineering, Buffalo, N. Y., died on Aug. 23. Mr. Fell was 52 years old and had been in the employ of the city for 28 years, and for 15 years in charge of sewer construction. He leaves a widow, two sons, one of whom is Lawrence J. Fell, also a civil engineer connected with the Buffalo Bureau of Engineering, and one daughter.

General Joseph Hayes, who was engaged in railway surveys in Iowa for what is now the Chicago, Rock Island & Pacific Ry., in the days before the Civil War, died on Aug. 19, aged 77 years. He graduated from Harvard University in 1855, and after a course in civil engineering took up railway work in the West. He served throughout the war in the Union Army and distinguished himself in many battles. After the war he again went into engineering work and for some years was engaged in mining operations in Colombia.

James Dyman Newton, Assoc. Am. Soc. C. E., Dean of the College of Engineering at Loyola University, Chicago, Ill., died in that city, on Aug. 8, aged 40 years. He received the degree of Mechanical Engineer from Cornell University in 1895. For a number of years he was an officer in the United States Revenue Cutter Service. After some time spent in mining work in New Mexico, he was appointed, in 1906, Assistant Professor of civil engineering at the University of Kansas. He had been at Loyola University about a year.

Joseph Le Conte Davis, Assoc. Am. Inst. E. E., Engineer in charge of design of direct-current railway motors of the Westinghouse Electric & Manufacturing Co., Pittsburgh, Penn., died Aug. 5. Mr. Davis graduated from the University of South Carolina in 1897 with the degree of Electrical Engineer. For three years he was a teacher of physics in the Bingham Academy High School in North Carolina and then entered the employ of the General Electric Co., at Schenectady, N. Y., where he was engaged in engineering work until 1904. Since 1904 he had been with the Westinghouse Electric & Manufacturing Co., at Pittsburgh, as a designer in the electric railway department. Mr. Davis was regarded as one of the most promising of the younger electrical engineers of the present day and his loss will be keenly felt by his many engineering associates.

John Krey, an engineer of many years' experience in various parts of America, died in Los Angeles, Calif., July 16. He was born in Danzig, Prussia, in 1857, and received his technical education at the Royal Technical School of Danzig. His first engineering work was in the Imperial Navy Yard. Later he went to South Africa and became a draftsman in the department of public works of Cape Colony, then draftsman with the Natal Government Rys., and later was engaged in railway surveys in Natal and Transvaal. Mr. Krey came to America in 1885 and was first employed with the Newport News & Mississippi R.R., later going West, he served in the engineering departments of the Northern Pacific and Duluth & Iron Range Rys. From 1891 to 1893 he was Municipal Engineer for West Duluth, Minn., but soon returned to railway work as Assistant Engineer of the Duluth Transfer Ry. A few years later he was appointed Assistant Engineer in the United States War Department and had charge of surveys for the proposed Lake Superior-Mississippi River Canal; he also had charge of breakwater construction at Lake Superior ports. In 1901 he became Division Engineer of the New York Central & Hudson River R.R. in charge of construction of the Weehawken terminal wharves, opposite New York City. In 1905 he had charge of the Alborn branch of the Duluth, Missabe & Northern Ry., in Minnesota. Later he served as Chief Engineer for the Katalla Co. in the building of the Copper River & Northwestern Ry. in Alaska. In 1907 he was again appointed Assistant Engineer in the War Department and was engaged in waterway improvement work in Virginia and Maryland and in the canalization of the Cape Fear River, North Carolina, until recently, when he went to Los Angeles under leave of absence, on account of his health. [Contributed by R. W. Moss, New Milford, Penn.]

Engineering Societies

COMING MEETINGS

INTERNATIONAL CONGRESS FOR TESTING MATERIALS.
Sept. 2-7. Sixth congress at New York City. Secy. of organizing committee, H. F. J. Porter, 29 West 39th St., New York.

INTERNATIONAL CONGRESS OF APPLIED CHEMISTRY.
Sept. 4 at Washington.
Sept. 6-13 at New York City. Secy., Bernhard C. Hesse, 25 Broad St., New York.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 7-10. Annual meeting at New York City. Secy., Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 9-12. Annual meeting at Hot Springs, Va. Asst. Secy., Walter Neumuller, 55 Duane St., New York.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 10-13. Annual meeting, Buffalo, N. Y. Secy., L. C. Ryan, C. & N. W. Ry., Sterling, Ill.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 10-13. Annual convention at Denver, Colo. Secy., A. P. Dane, B. & M. R.R., Reading, Mass.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 16-19. Annual convention at Niagara Falls, Canada. Secy., P. S. Millar, 29 West 39th St., New York.

PACIFIC COAST GAS ASSOCIATION.
Sept. 17-19. Annual meeting at San Diego, Calif. Secy., John A. Britton, 445 Suter St., San Francisco, Calif.

NEW ENGLAND WATER WORKS ASSOCIATION.
Sept. 18-20. Annual convention at Washington, D. C. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 18-20. Annual meeting at Washington, D. C. Secy., Selskar M. Gunn, 755 Boylston St., Boston, Mass.

INTERNATIONAL CONGRESS ON HYGIENE AND DEMOGRAPHY.
Sept. 23-28. Fifteenth congress at Washington, D. C. Secy., John S. Fulton, Senate Annex, Washington, D. C.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
Sept. 24-26. Annual convention at Buffalo, N. Y. Secy., R. D. Moldenke, Watchung, N. J.

INTERNATIONAL ASSOCIATION FOR THE PREVENTION OF SMOKE.
Sept. 25-28. Annual convention at Indianapolis, Ind. Secy., R. C. Harris, City Hall, Toronto, Ont.

NATIONAL IRRIGATION CONGRESS.
Sept. 30-Oct. 3. Annual meeting at Salt Lake City, Utah. Secy., Arthur Hooker, Salt Lake City.

AMERICAN ASSOCIATION FOR HIGHWAY IMPROVEMENTS.
Sept. 30-Oct. 5. Annual convention at Atlantic City, N. J. Secy., G. E. Pennybacker, Jr., Colorado Bldg., Washington, D. C.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 7-11. Annual convention at Chicago, Ill. Secy., H. C. Donecker, 29 W. 39th St., N. Y. City.

RAILWAY SIGNAL ASSOCIATION.
Oct. 8-11. Annual Convention at Quebec, Canada. Secy., C. C. Rosenberg, Times Bldg., Bethlehem, Penn.

AMERICAN RAILWAY BRIDGE & BUILDING ASSOCIATION.
Oct. 15-17. Annual convention at Baltimore, Md. Secy., C. A. Lichty, 226 Jackson Blvd., Chicago, Ill.

AMERICAN GAS INSTITUTE.
Oct. 16-18. Annual meeting at Atlantic City, N. J. Secy., Geo. G. Ramsdell, 29 West 39th St., New York.

ASSOCIATION OF RAILWAY ELECTRICAL ENGINEERS.
Oct. 21-25. Annual meeting at Chicago, Ill. Secy., T. A. Andreucetti, C. & N. W. Ry., Chicago, Ill.

American Institute of Mining Engineers—A joint meeting of the Institute with the Section of Mining and Metallurgy of the International Congress of Applied Chemistry will be held on Sept. 6-7, at New York City.

Institute of Operating Engineers—The second annual meeting of the Institute will be held at New York, on Sept. 6-7. Among the papers to be given are the following: "The Steam Engine Indicator," by C. S. Foster; "Electric Drive vs. Steam Drive," by J. F. Carman; "Plant Operation," by A. B. Reinhart, and "Keeping Tabs in Industrial Plants," by P. Lundeby. The Institute secretary is Hubert E. Collins, 29 West 39th St., New York.

American Institute of Electrical Engineers—Arrangements have recently been made by the Institute for a mutual exchange of membership privileges between itself and the leading electrical engineering societies of Great Britain and Europe. The substance of these arrangements is that visiting members, upon presenting proper credentials from their home society, shall have all the privileges of membership in the society visited, for a period of three months. It is expected that this plan will prove of benefit to the various societies involved, as well as of help to individual members who make use of it.

Up to the present time the following societies have signified their willingness to enter into this agreement: Institution of Electrical Engineers (Great Britain), Verband Deutscher Electrotecniker (Germany), Societe Internationale des Electriciens (France), Associazione Elettrotecnica Italiana (Italy), Kroninlijk Instituut van Ingenieurs (Holland), and Association Suisse des Electriciens (Switzerland). The Svenska Technologfereningen (Sweden) will consider the matter at the fall meeting of its governing body. Details and necessary credentials can be obtained from the Institute secretary, F. L. Hutchinson, 33 West 39th St., New York.